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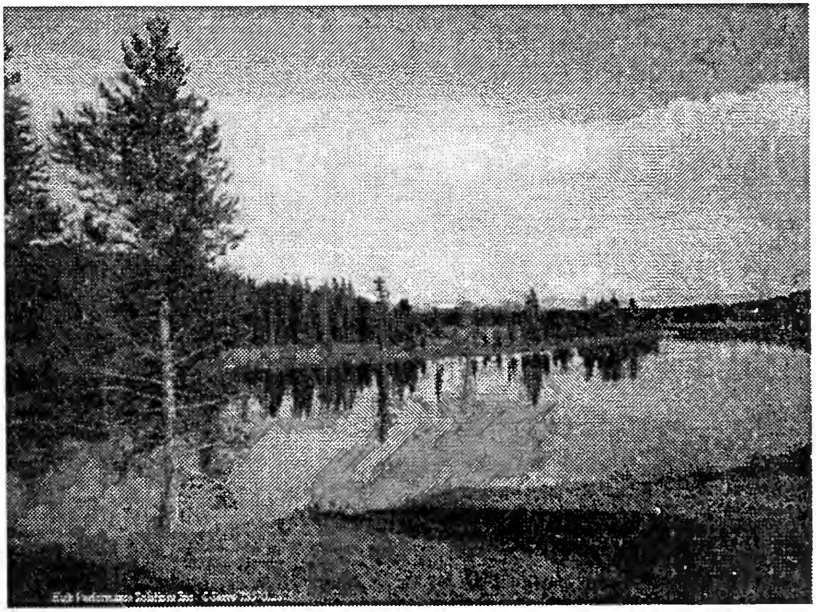
STORMWATER MANAGEMENT PRACTICES PLANNING AND DESIGN MANUAL



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STORMWATER MANAGEMENT PRACTICES

PLANNING AND DESIGN MANUAL

Report prepared for:

Environmental Sciences & Standards Division
Program Development Branch
Ontario Ministry of Environment and Energy

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Preface

The "state-of-the-art" of stormwater management has been rapidly evolving over the past decade. This manual is one step in this evolutionary process. The intent of the manual is to incorporate the experience of the past and advance/standardize current practice.

The manual provides technical guidance to professionals who are involved in the planning, design, and review of stormwater management practices. **It is important that this manual be viewed as a tool to assist in making decisions and not as a cookbook or rulebook for stormwater management solutions. The designer is solely responsible for decisions which are made with respect to stormwater management for any given site.**

Although the manual provides practical, specific guidance, there must be flexibility to account for site specific conditions. **Stormwater management solutions are site specific, and this must be recognized when applying the guidance which is provided in the manual.** Site specific conditions/characteristics will govern over the guidance provided in the manual.

Innovation must not be stifled by the manual. Where the designer can show that alternate approaches can produce the desired results, such designs will be considered for approval. There is a need for innovative designers to develop better designs and for reviewing agencies to encourage innovation by showing flexibility in applying agency criteria.

Acknowledgements

This report was prepared for the Ontario Ministry of Environment and Energy by Marshall Macklin Monaghan Limited. The principal writer of the document was Mr. G. Bryant of Marshall Macklin Monaghan Limited. The project officer was Mr. J.C. P'ng of the Program Development Branch of the Ministry of Environment and Energy. There are many sections which were collaborated on, or written by the multi-disciplinary team members. The following people and organizations are gratefully acknowledged for their assistance in the preparation of this document :

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The guidance of J. Mayes and M. Plewes of M.O.E.E is also gratefully acknowledged. The project advisory committee had their own distribution list within their agencies for review of the manual. These people are gratefully acknowledged for their helpful comments. In addition, people who participated in the Stormwater Guidelines and Technical Criteria Workshops are acknowledged for their contribution.

Executive Summary

A significant conceptual shift now is taking place in urban design; that is a movement away from "flat-earth planning" pushing nature into geometric forms, to a more environmentally sensitive "ecoplanning", where man and nature are mutually accommodated (Dorne and Rich, 1979). This statement, published 15 years ago, is a reminder that attitudes towards urbanization and the environment have been changing over the last several decades. Although this statement gives the impression that man is separate from nature, it was nonetheless visionary for its time. This Planning and Design Manual represents change; a change in the way development should be planned, a change in the way a development layout is determined, and a change in the way that stormwater management is dealt with in urbanizing areas.

Asking me to comment on the providence of nature is like asking a dog to comment on the brain of (Sir Isaac) Newton (Darwin, 1860). As our understanding of the world has evolved, we have found that the everything is inter-connected. We have recognized that single objective-oriented solutions usually cause more problems than they solve. This recognition has led us to the ecosystem approach for development through Watershed and Subwatershed Planning. It is no longer sufficient to plan parts of development in isolation. Today's development involves a multi-disciplinary approach with planners, engineers, biologists, geologists, and hydrogeologists. This integration extends to stormwater management on a subdivision/site level. This manual stresses an integrated approach with respect to stormwater. Individual stormwater management measures may be proposed for different objectives, however, the cumulative effect of all the measures must be determined.

The manual was prepared to provide a holistic approach to stormwater management, beginning at the watershed and subwatershed level, and extending to the subdivision/site plan level. There are seven main components of the manual which deal with stormwater :

- Watershed/Subwatershed Planning
- Subdivision/Site Planning
- Stormwater Management Practices Design Guidance
- Stormwater Management in the Absence of Subwatershed Planning
- Maintenance of Stormwater Management Measures
- Capital and Operations & Maintenance Costs
- Reviewer's Checklist

The manual is also available in a computerized information system format which can be used

as an educational / training tool.

The watershed/subwatershed planning chapter deals with the measures which are evaluated at a watershed/subwatershed scale to address stormwater management. These involve :

- management options and by-laws
- programs and projects
- the formulation and evaluation of urban stormwater management practices on a watershed/subwatershed scale

A methodology for the formulation and evaluation of urban stormwater management measures is presented in this chapter.

The subdivision/site planning chapter extends the ecosystem approach from watershed planning to the actual layout of the development. A methodology is provided to determine an appropriate development layout given the specific physical characteristics of the site in question.

Chapter 3 provides guidance on the design of individual stormwater management measures. This chapter is divided into 4 major sections :

- Stormwater Lot level Controls
- Stormwater Conveyance Controls
- End-of-Pipe Stormwater Management Facilities
- Integration of Stormwater Management Objectives

A preferred methodology is provided in this chapter which indicates that stormwater lot level controls must be evaluated first, then stormwater conveyance controls, and finally end-of-pipe stormwater management facilities. Lot level controls are primarily oriented towards maintaining the hydrologic cycle and are based on the premise of controlling problems at their source. Conveyance controls recognize that the timing of stormwater runoff, and what happens to stormwater as it is being conveyed to a receiving water can have a major impact on water quality, flooding, erosion, and groundwater recharge. End-of-pipe SWM facilities are the more traditional wet ponds and wetlands, etc., and reflect the "convenient" (although not optimal) solutions for stormwater management.

It is likely that a combination of these types of stormwater management measures will be required for any one site. Each of these measures will have an impact on water quality,

flooding, erosion, and groundwater recharge. The latter part of Chapter 3 provides guidance on how to assess the cumulative effects of the stormwater management measures on achieving water management objectives.

The fourth chapter, Stormwater Management in the Absence of Subwatershed Planning, reflects the need to apply an ecosystem approach to stormwater management planning, even if a watershed/subwatershed plan has not been prepared. The inclusion of this chapter is not intended to endorse the planning of development without watershed/subwatershed planning. Watershed/subwatershed planning is the only means of assessing the cumulative impacts of stormwater management measures on a broad scale and the preparation of these plans is strongly endorsed. The preparation of chapter 4 recognizes that there are areas where a watershed/subwatershed plan will not be prepared, at least for the foreseeable future. Proposed developments in a watershed with little or no widespread development pressure generally fall into this category. In these circumstances, guidance is provided in order to ensure that the stormwater management plan is holistic in nature recognizing the limitations of this approach.

The chapter on maintenance (Chapter 5) reflects the need to ensure that stormwater management measures operate on a long term basis. Although chapter 3 stresses that stormwater management measures should be designed to facilitate maintenance, convenience is not the primary design criteria for stormwater management. The stormwater management measures cited in this manual support the treatment train approach, which invariably, will result in higher maintenance requirements than purely end-of-pipe solutions. The desire for maintenance free solutions is one reason why water resources degradation has occurred in the past.

Capital and operations and maintenance costing information is presented in Chapter 6. The cost of stormwater management solutions must be assessed recognizing the limited economic resources of the province. The maintenance cost associated with a stormwater management solution is especially important recognizing that the measures must be maintained in perpetuity. The intent of this chapter is not to discourage the implementation of stormwater management measures, but rather to ensure that the most cost effective solution is implemented for the objectives to be achieved. Although some may feel that these stormwater costs are an additional burden to society, it must be remembered that the cost of not implementing these measures is far greater (Great Lakes Cleanup, Remedial Action Plans, dredging of the Keating channel, Don River cleanup costs, loss of tourism, loss of sports fisheries, etc.).

In recognition of the size of this manual, and the amount of guidance provided herein, a chapter is provided (Chapter 7) which contains a checklist of the key design parameters/conditions for

the various stormwater management measures covered in the manual.

This manual is the beginning, not the end. Throughout the preparation of the manual, it was recognized that stormwater management is evolving. This evolution would continue, irrespective of whether this manual was finished or not. The purpose of this manual is not to solve the world's problems, but to assist the evolution of stormwater management planning and design. The manual provides a framework to create the stormwater management solutions of tomorrow.

We do not have all of the answers. Much of the stormwater management decision making today is subjective. In this regard the manual strives to provide a level playing field for the province and to eliminate some of this subjectivity. There are no stormwater management experts, however, we are all still learning. **It is important that designers/reviewers treat this manual as a tool to assist them, and not as the definitive guide to stormwater management.** There are many site specific issues which affect development and stormwater management planning. It is important that this manual be used to assist professionals in making decisions and not used as a cookbook for stormwater management solutions. The designer is solely responsibility for the stormwater management decisions which are made for any particular site.

The influence of humans on the water cycle is a function of demographic development, cultural heritage, economic distribution, and social circumstances. The problems of pollution are related to the very roots of society in the broadest and most basic sense. It is important for the water profession to realize this when developing technologies. A holistic approach reveals that poor water quality is the consequence of a multitude of problems, and better technology may improve water quality but cannot solve those problems." (P. Harremoes, 1992). The degradation of our water resources is not the result of one influence, such as stormwater, but is the result of the lifestyle which we have chosen. It is caused by the cars we drive, the fertilizers/herbicides we use, the sediment we flush into the catch-basins, and our constant desire for convenience (ie. no wet back yards, extra parking lanes, flat grades for driving, no ditches, etc.). The stormwater management measures (ie. subdivision/site planning, lot level and conveyance SWMPs) presented in this manual address the symptoms, but not the cause of water resources degradation. We must recognize that we (humans) are the cause of water resources degradation, and that the success of our efforts to protect/enhance the environment will depend on the extent to which we, ourselves, are willing to change.

Terminology

It is clearly evident that the term Best Management Practice (BMP) has not been used throughout this manual. The overuse of the term has created confusion since it has become synonymous with everything and anything which enhances the environment. In addition, there is considerable confusion across disciplines since the term BMP is used to represent quality control practices in many different fields of work.

Therefore, terminology which is specific to the stormwater management industry was used in the preparation of the manual. The following list of terminology provides the basis for concepts discussed in the manual :

Stormwater Lot Level Controls

These stormwater management practices represent measures which are implemented at the lot level (soakaway pits, flatter grading, etc.)

Stormwater Conveyance Controls

These stormwater management practices represent the conveyance systems used to transport stormwater runoff from the lots to the receiving waters (pervious pipes, grassed swales, etc.)

End-of-Pipe Stormwater Management Facilities

These stormwater management practices represent the common urban stormwater management measures used to service numerous lots or whole subdivisions (wet ponds, wetlands, infiltration basins, etc.)

The foregoing three terms are generically called **Urban Stormwater Management Practices** or **SWMPs** in an acronym format.

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1.0 WATERSHED/SUBWATERSHED PLANNING

1.1 Introduction

In June 1991, the Ministry of Environment and Energy published a report entitled "Stormwater Quality Best Management Practices". The report documented current experience with structural and vegetative Stormwater Management Practices (SWMPs) and concluded that they should be implemented in conjunction with new urban development and redevelopment.

Guidance, and a procedure for selecting appropriate SWMP types was provided. The report stated however, that "integrated watershed planning is the preferred means of defining uses of the receiver and hence the basis for SWMP selection". Recognition of the importance of watershed and subwatershed based planning has continued to grow since the release of the 1991 study.

In May 1992, the Ministry of Environment and Energy initiated the development of a Stormwater Management Practices Planning and Design Manual. Originally conceived as a technical manual to provide guidance and criteria for the design of structural and vegetative SWMPs, the scope of the project was expanded in recognition of the importance of watershed and subwatershed planning. While structural and non-structural SWMPs will be an important tool in protecting the natural environment from the impacts of land use change, they will form only part of a more holistic strategy which must be aimed at preserving and protecting the natural function and interlinked systems of the ecosystem.

The intent of this chapter is to provide a brief discussion of watershed/subwatershed planning and the process which will lead to, among other things, the definition of requirements which will govern SWMP selection and design for urban development. The concepts related to watershed planning and SWMP planning have evolved rapidly during the last several years. The management options available to the watershed planner encompass management programs (eg. salt, top soil controls), land use (eg. buffers and linkages), in-stream enhancements (eg. geomorphological and habitat alterations, stream bank revegetation) as well as the urban lot level controls and end-of-pipe SWM facilities which are usually associated with new development or redevelopment. As a result, the chapter discusses the range of management options which may form part of a watershed or subwatershed plan, in order to provide an overall context. The greatest emphasis and discussion however are related to outputs from the planning process directed toward guidance for urban SWMPs. This discussion acts as an introduction to subsequent technical and design chapters.

1.2 Existing Guidelines

Those familiar with water management planning will recognize the terms Watershed Plan, Subwatershed Plan and Stormwater Management Plan. It is accepted, as a general rule, that these plans will provide water-related environmental input to municipal land use plans,

specifically, Official Plans, Secondary Plans and Plans of Subdivision. A typical relationship between watershed and municipal planning is illustrated in Figure 1.1

For more information on watershed planning, the reader is directed to the following :

> "Water Management on a Watershed Basis : Implementing an Ecosystem Approach", June 1993, Ministry of Environment and Energy and Ministry of Natural Resources.
>
> "Subwatershed Planning", June 1993, Ministry of Environment and Energy and Ministry of Natural Resources.
>
> "Integrating Water Management Objectives into Municipal Planning Documents", June 1993, Ministry of Environment and Energy and Ministry of Natural Resources.

These three reports were released in June 1993 to guide watershed planning in the province. They are intended to be used for voluntary application in land use and resource management decisions throughout the province.

These documents describe a process designed to assist agencies in working together to balance the province's social, environmental and economic needs. This process draws upon recent experiences and progress in several communities in the integration of water resources planning with municipal land use planning.

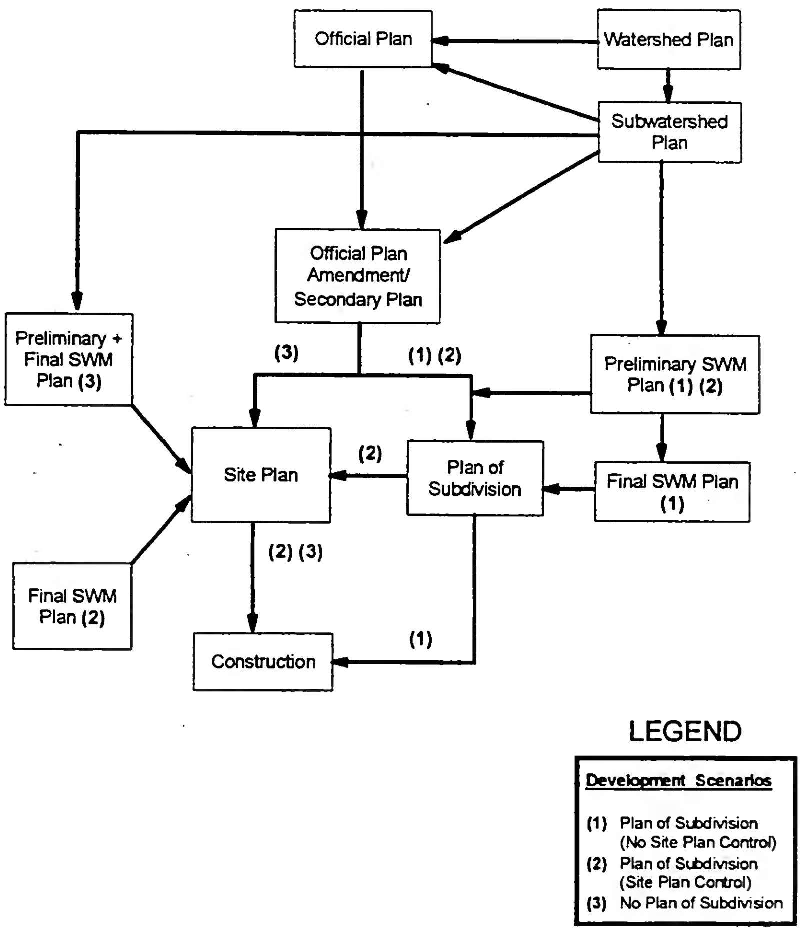
1.3 Intent of Watershed/Subwatershed Plans

Watershed and subwatershed plans provide important information into the land use decision-making process for the use and management of water and land. It is the intent of these plans to demonstrate mechanisms which compatibly integrate natural systems with changing land use.

It is recognized that sound environmental management is important for the economic health and sustainability of successful communities. Recent reports by the Royal Commission on the Future of the Toronto Waterfront, "Watershed" (1990) and "Regeneration" (1992), promoted the concept that watersheds serve as natural and logical boundaries for modern approaches to urban environmental and land use planning.

Watershed planning and land use planning consider the same environmental issues, but from differing viewpoints and at different levels of detail. A land use planning decision for a site specific development can influence many watershed management and land use planning issues. The input of environmental objectives and management recommendations to the land use planning process at appropriate stages should promote informed decision-making, which will in turn lead to greater efficiency and effectiveness of both processes.

Figure 1.1 Watershed and Municipal Planning



Watershed planning treats the entire ecosystem as an integrated collection of smaller drainage basins or subwatersheds. An action or change in one location within a watershed can affect the other natural features and processes that are linked by the movement of the water. Integrated watershed and municipal planning, together with improved management practices, represents a significant step along the path to a fully functional ecosystem approach.

1.4 Implications of Scale

Watersheds vary tremendously in size and it is often unclear what is meant by a Watershed Plan and a Subwatershed Plan. Clearly the latter could be a sub-element of the Watershed Plan, but for medium and large watersheds the subwatersheds themselves may be of substantial size and may contain a number of important tributaries. Should extremely detailed subwatershed plans be completed for tributaries that are many times the size of some watersheds ?

The answer lies recognizing Watershed/Subwatershed planning as a **process**. The process requires that we develop an understanding of the important biological and physical elements and their inter-dependencies and linkages and that ultimately we translate this understanding into management decisions and direction to developers wishing to proceed with individual plans of subdivision. So long as both these requirements can be met, it does not matter whether one comprehensive plan is completed or whether the process is divided into an overall plan and a series of sub-plans. Where multiple plans are carried out, it will not matter where one starts and the other begins so long as a complete understanding is developed and used in decision making. The demarcation between watershed plans and subwatershed plans may be determined on a case by case basis. The scale or size of the area will usually be used to determine the scope of each plan. Further discussion of watershed/subwatershed planning and examples of some of the approaches which have been used in recent studies are provided in Appendix A.1.

1.5 Watershed/Subwatershed Planning Procedure

Both watershed and subwatershed plans are prepared using the same procedure, although their focus and level of detail are very different. The interim provincial guideline on Subwatershed Planning suggests that a two-phase process will often be workable but that phasing should be tailored to meet local issues and concerns. The overall approach to watershed/subwatershed planning, as outlined in the interim document "Subwatershed Planning" (1993) is reproduced in the following two sections.

Overview

The technical studies in support of subwatershed plans should be flexible, cooperative and practical in order to successfully integrate watershed management and land use planning.

Typically, a team of experts undertakes technical work on behalf of the Steering Committee. Here are some key features of this work:

Flexible - Each subwatershed study needs to be tailored to specific subwatershed issues and local municipal concerns. It should also recognize the status and recommendations of watershed plans where available.

Multidisciplinary - These studies require environmental, planning and engineering expertise to provide analysis of a wide range of environmental issues and development options.

Integrated - An understanding of all components of natural and man-made environments affecting the integrity of natural systems is a critical component of these studies.

Time Saving - Subwatershed plans can reduce the time spent on site-specific plans, e.g., stormwater management, review and approval processes, by providing a "blueprint" of requirements for all subsequent works. In this way, the number of small site-specific plans can be reduced and addressed for refocusing, and duplication of effort avoided.

Staging Plan Development

It has been suggested that the best way to integrate technical components of the plan with land use planning decisions is to carry out plan development in stages, or phases, so that the plan unfolds consistently and in conformity with real conditions, as more information is gained from technical assessments, and can be incorporated into key decision points or mechanisms in the land use planning process. This approach is not intended to lengthen the time frame of plan development, but rather to enable participants to collectively make decisions about the subwatershed at key points throughout the evolution of the plan. It can also enable some studies or activities to be undertaken when complete funding and/or support is not immediately available.

This document (Subwatershed Planning, 1993) discusses a two-phased approach. However, more phases could be added in order to respond to local concerns and needs. In some situations, for example, because of resource limitations, an initial phase could be simply the gathering of background data, and establishment and preparation of terms of reference.

Phase 1 will:

- outline the location, extent, sensitivity and significance of all components of the natural systems
- identify land/water linkages and processes
- identify factors and influences that are important to the integrity of various existing or desired components of the environment

- identify watershed and subwatershed goals, objectives and targets

- identify opportunities for protection, enhancement, rehabilitation and development

- identify monitoring needs

- identify plan review and update schedules

The complexity of Phase 1 work depends on whether watershed plans or other relevant environmental planning studies have been completed. For example, watershed and subwatershed objectives and targets may already be established; information on natural features to be protected may already exist in environmental or green space planning studies. Phase 1 of a subwatershed plan should incorporate or complement, not duplicate previous relevant work. If no previous studies are available, some aspects of the watershed plan could be done as part of Phase 1 activities.

Phase 2 will develop a plan that will recommend:

- areas to be protected, enhanced and rehabilitated

- various types/intensities of proposed development

- management practices for open space areas

- best management practices and designs for the management of the quantity and quality of surface water and ground water

- an implementation strategy to guide development, those responsible for designing and building recommended works at what time, and responsibilities and requirements for cost-sharing, future studies, monitoring and maintenance

1.6 Preparing the Watershed/Subwatershed Plan

1.6.1 Project Initiation

The initial steps in preparing a watershed or subwatershed plan include establishing the need for the plan, organizing the participation of a multi-agency steering committee, defining the extent of the study area and the planning approach to be followed, specifying study goals and objectives, preparing a terms of reference and selecting a consultant. The interim provincial guidelines on "Subwatershed Planning" (1993) provide a comprehensive discussion of the initial project steps.

1.6.2 Early Project Stages

Once development of a plan has been initiated, a series of well defined stages are typically carried out. Depending on the complexity of the watershed/subwatershed plan these may be treated as phases, with specific reporting and public involvement milestones. The purpose of theses early stages is to develop a strong understanding of the key functions and interactions which influence the integrity and health of the ecology. Typically, the process of developing this understanding involves a thorough background review, usually including preliminary field reconnaissance and public consultation, in addition to the collection and review of existing reports and data sources. Synthesis of available information across disciplinary lines is carried out in order to identify the major interdependencies between the physical systems and the biological systems. Data deficiencies which are important enough to limit decision making are identified and a refined and focused work plan is prepared. Completion of the background stage is normally followed by a technical stage in which field studies are conducted, data are collected and evaluation tools are formulated and calibrated.

A more detailed discussion of the elements of these early project stages is provided in Appendix A.2. The information presented provides one example of the kinds of investigations and techniques which are may be undertaken. Additional and alternative techniques are available and may be applied in specific cases. Care must be taken not to blindly employ the various techniques discussed without regard to the actual issues and needs of the watershed/subwatershed study.

Once the early project stages are complete, the process of plan formulation begins. The overall process, as discussed in the following sections, is illustrated in Figure1.2.

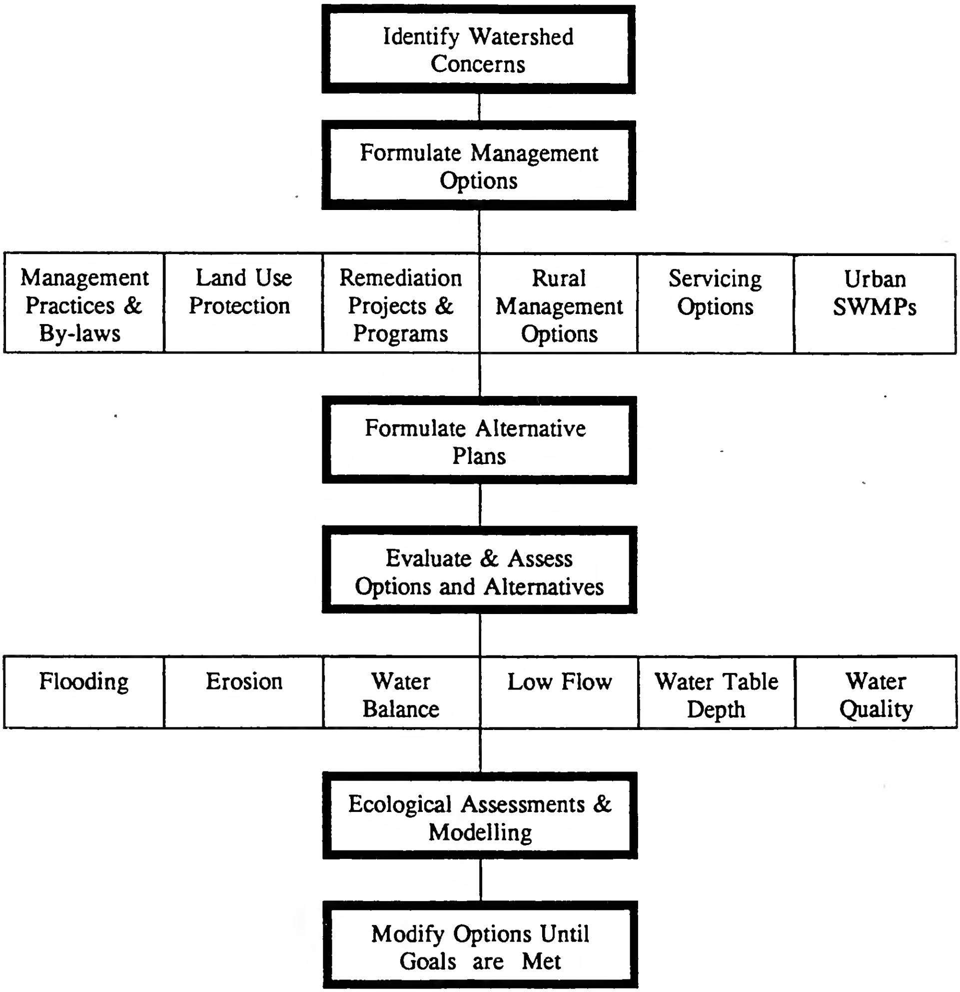
1.6.3 Formulation of Management Options and Alternative Plans

With the completion of the background review and subsequent technical studies to fill in major information gaps, the best possible "picture" of the watershed/subwatershed and its key systems and linkages will have been prepared. The next step is to formulate a series of management options and alternate management plans, based on the understanding of the watershed/subwatershed and the goals and objectives which have been established. The range and different types of management options will normally cover a wide spectrum, because the goals are usually far-reaching, calling for preservation, protection, enhancement and rehabilitation.

All of these management options are necessary to a watershed or subwatershed plan because of the variety of ways that human use can affect ecological health.

Figure 1.2

Plan Formulation Process



Types of Management Options

Within the context of this manual, the major focus is on the urban SWMPs, specifically, on the lot level, transport, and end-of-pipe controls which accompany modern urban development. Some of the other management options do influence urban development, and combinations of all options (both urban development oriented and other) must be taken into account when evaluating the alternate plans. A brief discussion is therefore provided for the different types of management options normally considered in a watershed/subwatershed plan. The implications to urban development are noted where applicable.

Management Practices and By-Laws - This class of management options includes actions which effect programs carried out by municipalities, conservation authorities or provincial agencies. Examples include road salt management, use of herbicides and pesticides, flood and fill regulations, and municipal top-soil, ravine protection or tree by-laws. In many cases management programs will already be in place on a generic basis. Management practices or by-laws are usually recommended when technical studies have noted a specific problem.

As an example, rising chloride levels in a stream near a major arterial or a particular subdivision, may warrant review of a municipality's salt application policy (eg. mixtures, snowfall threshold prior to salting, salting of only designated hazard areas). This management practice would supersede general municipal policy within the subwatershed. Similar management practices might be applied to municipal use of herbicides and pesticides on public lands. Again, such management practices are normally recommended only when there is evidence of a problem or other reason for concern.

By-Laws represent a special class of management practices. They are usually passed to give the municipality a better enforcement capability and more timely control over matters involving construction and development practices. In particular, Top Soil By-Laws may require a permit to remove topsoil and may regulate stockpiling and rehabilitation of the land. The adoption of such a by-law is often recommended when there is a history of land clearing without adequate sediment control measures. The by-law approach is often considered preferable to reliance on sedimentation and erosion control plans because it applies to all lands and therefore can address the problem of site clearing prior to development applications being made. Similar advantages exist for ravine protection and tree preservation by-laws.

Urban development must comply with the provisions of regulations and by-laws, which may govern the amount of developable land and the manner in which it may be developed. In general, this class of management options has no impact on stormwater management plans and their associated SWMPs, except in some terms of how the sediment and erosion control (during construction) are carried out.

Land Use Protection - This group of management options includes provincial policy and municipal land use restrictions placed on lands to be protected because of their intrinsic value or their importance to the overall health of the watershed/subwatershed ecosystem. Land use restrictions will normally be placed on significant wetlands, areas of natural and scientific interest (ANSIs) environmentally significant areas (ESAs), buffers around wetlands and along stream corridors. They may also be placed on lands that provide linkages and corridors between natural areas or which serve a significant natural function (eg. recharge, seasonal spawning area, food supply, etc.) vital to the protection of the watershed/subwatershed ecology.

These types of management options can of course have a major effect on urban development proposals by limiting the amount of land which is available for development. They may also provide opportunities (in some cases stormwater facilities may be allowed within a buffer area) or constraints (development form may have to be innovative in order to preserve natural drainage features). It should be recognized that the performance criteria specified for urban developments are evaluated in the watershed /subwatershed plan in a manner which accounts for the benefits of protecting lands having significant natural value or function. More stringent criteria would normally be needed if these areas were not retained in an undeveloped state.

Remediation Projects and Programs - This type of management option involves specific projects or programs undertaken to remediate, rehabilitate or enhance portions of the watershed/subwatershed where a problem or limitation has been identified. Examples of this type of management option may include revegetation of riparian corridors, geomorphological and stream habitat projects (eg. creation of pool and riffle areas or spawning areas, using natural techniques), channel rehabilitation (eg. replacement of concrete channels with natural channels) or removal of obstructions to flow or fisheries movement.

In most cases, specific projects will be proposed as a result of the geomorphological or aquatic habitat inventories or surveys (together with subsequent analyses) conducted as part of the subwatershed plan. In virtually all cases the projects will be intended to create stable natural channels which will improve available aquatic habitat, or otherwise benefit the ecology. These projects will generally rely on natural channel design techniques. Guidance on natural channel design techniques is provided in "Natural Channel Systems : An Approach to Management and Design, Development Draft" (Ministry of Natural Resources, 1994).

Generally, these types of management options are undertaken at locations which are on publicly owned land or on lands which are dedicated as part of the land development process (eg. because of land use restrictions). They are often recommended for reasons which are related to urban development and therefore must be evaluated in conjunction with the SWMPs associated with urban development. For example, the effects of urban SWMPs on temperatures and flow (flood and erosion impacts) must be accounted for in

evaluating remedial projects such as riparian revegetation and geomorphological improvements. Depending on the results of the combined assessment, additional requirements on the urban SWMPs may result. In some cases the use of natural channel design techniques to create stable channels will be evaluated as an alternative to traditional quantity control (for erosion control purposes).

Rural Management Options - While land use change and urban development are usually a concern in the development of watershed/subwatershed plans, existing land uses such as agriculture and aggregate extraction are also important. Management options specific to these land uses are normally developed as part of the plan. Examples include practices such as the use of filters strips or buffers separating fields from watercourses, or fencing of livestock from streams, or specification of dewatering practices for aggregate operations. These management options typically have no implications to urban development, except in terms of the watershed's capacity to support human use while sustaining a healthy ecosystem.

Servicing options - Servicing options include the management actions which govern the provision of municipal services. This may include the regional stormwater infrastructure and provision of sanitary services or water supply. These options will not normally have a major influence on specific developments so long as the provision of a particular service is not limited by the watershed/subwatershed plan (eg. water supply from a communal well, groundwater interference by a servicing trench, development on septic systems). Instances may arise however where the servicing options do have implications (eg. service routing to avoid sensitive areas or the requirement for regional stormwater facilities) which must be accounted for in the design of urban developments.

Urban SWMPs - This class of SWMP includes various lot level, transport, and traditional end-of-pipe facilities as well as land use specific SWMPs. These SWMPs are integral to urban development in areas approved for development and are the focus of this manual. Chapter 3 (Stormwater Management Plan) provides guidance on the design of urban SWMPs.

A more detailed discussion of the selection of urban SWMPs for use in watershed and in particular, subwatershed planning, is provided in the following section.

Urban SWMPs

Watershed and subwatershed plans completed in recent years have all incorporated urban development scenarios which have stormwater quality, quantity and erosion SWMPs as an assumed part of the development form. In general, this has been done intuitively, based on experience, knowledge of the watershed's physical and ecological systems, and the goals and objectives of the plan. Normally, the procedure is to determine the management options to be used and the level of control to be required and then to test the performance of the options on

the key physical and biological systems of the watershed. Because of the scale of the study area, the lack of detailed planning information over large areas and the number of alternatives to be considered, representative SWMP forms (generally lumped) have been used together with a number of discreet levels of control (eg. 13 and 25 mm storms, 100 year event) to size the assumed facilities. In most cases the selection of a SWM facility type for testing is restricted to either an "infiltration" type or a "storage" type (usually a wet pond).

Once a framework of SWMPs is laid out, the long term impact of the facilities is tested, usually with continuous simulation models. These simulations provide input to subsequent evaluations relating to water levels, temperature, flood duration, flow velocity, etc. The groundwater system may or may not be explicitly considered in the evaluations, depending on the importance of this system within the particular watershed/subwatershed. Lot level and transport SWMPs are rarely modelled or tested directly (although assumptions may be made regarding post development infiltration) but recommendations for these are often incorporated into the plan in order to encourage developments which more closely approximate the natural hydrologic cycle.

While this approach works well in the development of the watershed/subwatershed plan, it can lead to difficulties when subsequent stormwater management plans are undertaken. If the proponent wishes to use a wetland rather than a wet pond, or a combination of SWMPs rather than a single type, the resulting operation and impact of the facility may differ from the assumptions made in the watershed/subwatershed plan. Longer flow discharge characteristics or higher/lower temperature discharge may occur. Ideally, the watershed/subwatershed simulations should be repeated when a different configuration of SWMPs is used, but this is rarely practical for budgetary reasons. If a proponent is required to use the SWMP types assumed in the watershed/subwatershed plan then the flexibility and innovation which these plans are supposed to promote will be stifled.

There is no easy answer to this problem. The likelihood of major differences between the watershed/subwatershed and development level assumptions can be minimized however, by incorporating a more explicit procedure for selecting SWMPs for testing at the subwatershed plan level. The following procedure is recommended:

1. Determine subwatershed concerns (water quality, flooding, erosion, fisheries)
2. Determine preliminary recharge estimates (water balance)
3. Identify SWMPs to address concerns (based on experience, or see Chapter 3, Section 3.6, Table 3.5)
4. Size SWM facility for water quality control based on desired level of protection and expected level of imperviousness in the urban development (refer to Chapter 4, Section 4.4.1 , Table 4.1, on end-of-pipe controls, for guidance).
5. Increase SWM facility active storage volumes and/or add SWMPs to provide for expected erosion and flood control volumes needed
6. Cost SWMPs (if scale of subwatershed permits)
7. Rank SWMPs based on experience, Steering Committee direction and cost (if scale of subwatershed permits)

8. Select SWMP Plan (single or combinations of SWMPs)
9. Configure simulation tools to reflect selected SWMP plan
10. Conduct evaluations and assessment as described in Section 1.7
11. Modify SWMP types and sizes, as necessary, in order to meet watershed or subwatershed goals

There are three key elements in this procedure. The first is the selection of the level of protection in Step 4. Each level shown in Table 4.1 of Chapter 4 represents a different level of suspended solids removal. As stated in that Chapter, the second level would normally be selected for most water courses. The highest level would be applied in areas deemed to be sensitive from an aquatic life perspective. The third level would be expected in areas with currently poor aquatic habitat, where there is no expectation that remediation or rehabilitation will take place. The last level would be applied only in remedial or retrofit situations where space restrictions limits the ability to provide better control. Clearly, determining where a stream fits into these levels is a part of the watershed or subwatershed planning process. The determination will require input from specialists on the subwatershed planning team and from both the Steering Committee and the public.

The second key element to this process is recognizing the differences in the storage requirements for the different SWM facilities and ensuring that the watershed/subwatershed modelling procedures correctly reflect the expected configuration of the SWM facilities. For the infiltration and dry pond SWM facilities the stated storage volume/hectare requirements are straight forward. For wet facilities (eg. wet ponds and wetlands) the listed storage volumes (in Table 4.1, Chapter 4) represent the total of permanent pool and active storage volumes. The active storage component is 40 m^3/hectare, regardless of the imperviousness assumed. In the design of wet SWM facilities it has been found that the permanent pool volume is the key to pollutant removal. In assessing the effects of SWM facilities on a watershed or subwatershed scale however, it is the discharge from the facilities which is important. Thus it is the active storage and its rate of drawdown or the frequency of overflow which must be modelled in detail.

The last important element in the process is in the ranking of SWMPs in Step 7. It is important that the Steering Committee members provide input regarding local or regional preferences for the SWMP types as part of the ranking process. Experience has shown that selection of SWMP type (at the individual development level) is often influenced by agency preference.

Alternative Plans and Cumulative Impacts

In order to evaluate the management options and different types of SWMPs it is necessary to formulate alternative plans which use various combinations of options. All types of SWMPs are used in the formulation of alternative plans. Situations may arise where a problem is attributable to a single source and can be addressed through a specific option. This will, however, be the exception rather than the rule. Most problems originate from a variety of sources and require a variety of management actions. Concerns for stream temperature impacts on fisheries may, for instance, require protection of groundwater discharge areas and sources, restoration of stream

canopy, creation of in-stream refuge areas, and use of particular stormwater SWMP types in urban development. No single option can solve the problem and it is necessary to examine the effects of the entire set of options. The individual options may be the responsibility of different groups, both private and governmental, and it is implicit in the evaluation and assessment that all parties will implement those options which are their responsibility.

There is a second important factor in the formulation of alternate plans for evaluation, and that is the assessment of cumulative impact. Land use change produces both direct and immediately observable impacts and more gradual impacts which may build up to critical levels as growth proceeds. A range of options aimed at addressing a particular problem may cease to be sufficient if too much development takes place. The alternate plans must therefore be evaluated not only against each other but also under a range of growth conditions.

While it is not possible to specify the alternate plans to be considered in individual watershed/subwatershed plans on a generic basis, the following list of scenarios represents an example of one set of alternate plans which might be considered. Clearly, the need for some or all of these alternatives will be dependent on the characteristics of the watershed/ subwatershed.

1) Evaluation of the existing condition (as a bench mark) or alternately a base alternative which applies basic regulatory requirements and constraints (eg. flood plain or hazard land constraints, significant natural areas to be protected, required buffers).

2) A preliminary scenario which includes management options which are not contingent upon urban development proceeding (eg. in-stream rehabilitation, remedial actions, restoration, etc.).

3) A scenario which considers management options for non urban areas (eg. agriculture, aggregate extraction, forest management) where these are important activities on the watershed/subwatershed. This scenario may be combined with 2) in some instances.

4) A servicing scenario (separate from development scenarios) which may include such things as development of municipal wells, trunk sewers or major arterial road networks.

5) Development scenarios. Several scenarios or alternative plans should be considered to examine different levels of growth.

The scenarios would normally be evaluated in a cumulative fashion with the management options which perform well being carried forward into subsequent alternatives. This allows the assessment of increasing levels of stress being applied to the watershed/subwatershed.

It is particularly important to consider the cumulative effect of urban development SWMPs on the watershed/subwatershed. While these SWMPs are designed to reduce sediment and contaminant impacts from urban runoff, they may alter the water balance, flow regime or

thermal regime of the water courses. Current practice is to seek to more closely maintain the natural hydrology of developing areas through the use of lot level controls to promote infiltration. This is a goal however and can almost never be completely achieved. The effect of extended discharge on flows, erosion thresholds, water levels (as they impact aquatic life or wetlands) must therefore be accounted for in developing the watershed/subwatershed plan. In some cases the cumulative effect of more and more SWMP discharges may lead to the need for limitations on urban growth.

1.7 Evaluation and Assessment

The evaluation and assessment of alternate options and plans involves a combination of qualitative and quantitative assessments. The recommended approach is to conduct a qualitative evaluation of all options initially. Some options cannot be evaluated in anything but a qualitative manner. For those which can be evaluated quantitatively, a variety of techniques are available. An evaluation methodology for SWMPs in a Subwatershed Plan is shown in Figure 1.3. This figure is an expansion of step 10 in the SWMP selection process for subwatershed planning (Urban Development SWMPs, page 13). It indicates the steps required to evaluate the effects of SWMPs on the physical and biological attributes of the stream system. Examples of some of the techniques used are provided below.

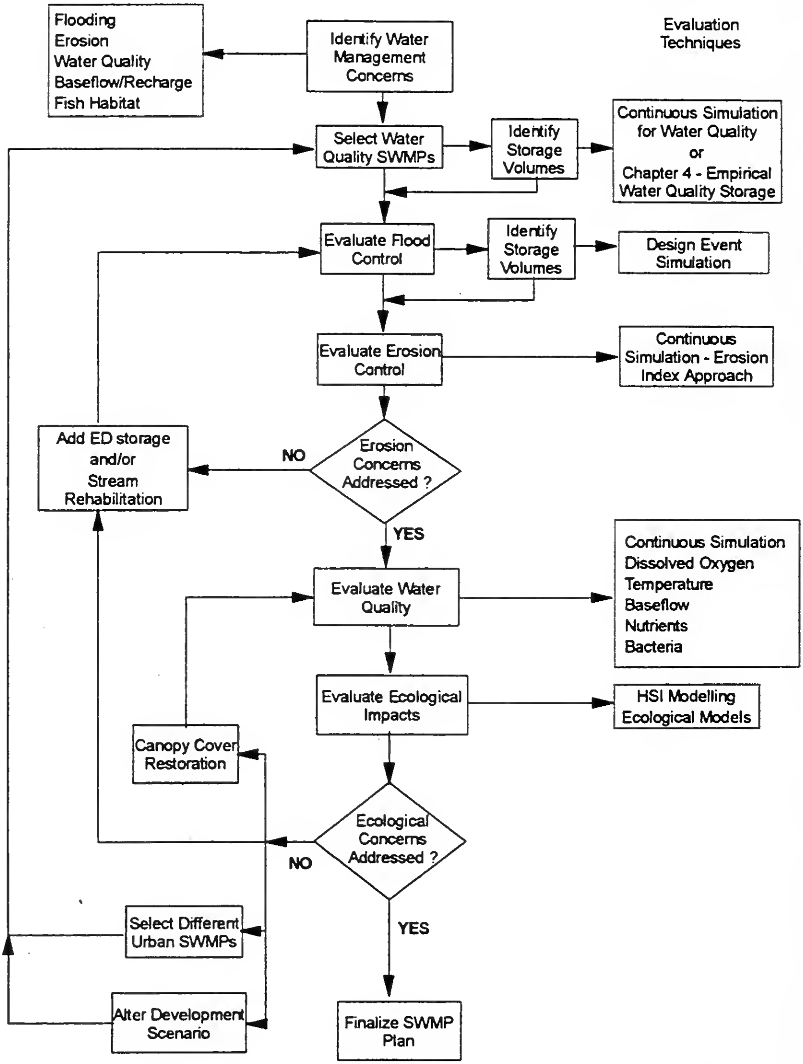
Flood Evaluations

The most common flooding evaluations and assessments conducted in watershed/subwatershed plans involve event modelling of infrequent precipitation/snowmelt events. The procedures and models used have become standardized in the consulting community. The principal questions to be answered from the watershed/subwatershed planning perspective are the level of quantity control required, in different areas, the extent of the regulatory flood lines vis a vis the stream valley and the opportunities for remediation. The assessments have to account for progressive growth on the watershed and it is often necessary to assess the effects of other proposed management options (eg. revegetation, rehabilitating channelized sections) on the anticipated flood elevations.

In addition to the flood assessments related to damage prevention and public safety, flooding assessments may be required in connection with the preservation/protection of terrestrial or wetland habitats. In such cases it is normal to use continuous hydrologic models so that changes in the depth and duration of seasonal flooding can be examined under various growth scenarios.

Evaluation of the performance and impacts of the urban SWMPs are a major part of flood evaluations. The watershed/subwatershed should indicate the approximate locations and level of control required for flood control, taking into account any other management options which are being considered (eg. removal of obstructions, restoration of channelized reaches).

Figure 1.3 SWMP Evaluation in Subwatershed Planning



Erosion Evaluations

The evaluation of increases in the stream's potential for bank erosion as a result of land use change normally involves continuous modelling of the surface flow regime so that changes in the duration and velocity of flow can be examined in relation to the stream's susceptibility (eg. soils, geomorphology). The modelling conducted for erosion assessments is done in conjunction with the modelling of water quality SWMPs so that the effect of the extended release from these facilities is synthesized with in-stream changes in flow and velocity. In many cases the level of control to be utilized for water quality SWMPs and the SWMP types to be employed are determined through a joint consideration of the fisheries related uses of the stream and erosion concerns. This may require adjustment of the assumed volumes of SWMPs (from those derived from Table 4.1, Chapter 4). The watershed/subwatershed plan should indicate the level of control required for erosion protection.

Water Balance and Low Flow Modelling

In many watershed/subwatershed studies the water balance is examined for the alternative plans so that there is an understanding of how the groundwater and surface water regimes will react to land use change or management options.

There are various models which can be run to simulate surface runoff on a continuous basis. In most of these models, very shallow groundwater flow (interflow) can be simulated, but there is no modelling package which allows a convenient approach to the modelling of the interaction between surface water and complex groundwater systems. It is possible to evaluate the effect of land use change on the low flow regime in terms of the quantity of flow, using these simple models but is not possible to evaluate the relative proportions of groundwater and surface water inputs, except in a broad way. It is noteworthy that some watershed/subwatershed plans have shown that low flow levels can increase after urban development even though groundwater contributions to the stream are diminished. This results from the extended release of stormwater runoff from urban SWMPs over a period of days.

In areas where the aquatic resources are particularly sensitive (eg. headwater areas, smaller spawning tributaries) efforts can be made to link and interface surface and groundwater modelling packages. The effort required is substantial however, and can usually only be undertaken if there is a substantial database available for the groundwater system.

Where a watershed or subwatershed plan specifies requirements for either lot level infiltration or end-of-pipe infiltration, the requirement should be stated as a target volume on a unit area basis, together with the assumed percolation rate for the soil. In conducting infiltration and low flow assessments the subwatershed planner will utilize broad infiltration capacities based on the soils of a large subcatchment. In practice, actual soils and their capacity to infiltrate runoff may vary markedly from site to site within the subcatchment. It may be impossible to achieve infiltration targets in some areas and possible to achieve more in other areas. The performance criteria specified by the subwatershed plan should recognize this and be stated so as to allow

flexibility and adjustment on a site by site basis, while seeking to maintain the overall subcatchment infiltration requirement.

Water Table

In cases where vegetative communities, particularly wetlands, are to be protected, the depth to the water table may be of crucial concern. In such cases groundwater modelling is generally required in order to assess the effect of changes in imperviousness as watershed growth occurs.

Water Quality Modelling

In most watershed or subwatershed planning evaluations, water quality modelling is carried out at two different levels, the first on a general contaminants basis, and the second on a parameter specific basis.

The general water quality modelling of contaminants is normally carried out using simple empirical or mass balance approaches, linked closely to shifts in the surface water/groundwater contributions to flow. Contributions of contaminants from developed or developing areas will normally be estimated based on standard literature values. The greatest emphasis is placed on suspended solids, with most chemical parameters being estimated using literature - derived correlations with suspended solids.

This form of water quality assessment is practical as a planning level evaluation of potential water quality changes caused by land use modification. More sophisticated modelling techniques are available but they can rarely be justified, due to the lack of a comprehensive database and inherent model limitations (eg. a model may deal with one area adequately, e.g. contaminant build-up and wash-off but be inadequate in the manner in which it deals with treatment, transport, or chemical processes).

The second level of water quality modelling which may be required involves a more directed approach to the assessment of key factors influencing specific uses or resources. Examples of this form of water quality assessment may include modelling of parameters such as temperature, dissolved oxygen, bacteria or nutrients (to assess trophic level). A variety of models or procedures are available to conduct specific assessment for these different parameters. In many cases, however, the models must be adjusted or augmented to reflect factors of interest (such as stream canopy in temperature modelling). In most cases, this form of water quality modelling is undertaken as an intermediate step, to produce data which is to be used in other assessments (eg. such as ecological models). This type of modelling therefore requires a high degree of cooperation between the modeller and the end user, so that meaningful results are achieved.

In general, urban development SWMPs are not modelled explicitly in terms of their sediment, chemical or bacterial discharge. Where standard stormwater quality control is desired facilities are sized according to accepted rules. The sizes of facilities specified in Table 4.1 (Chapter 4) for example are based on the removal of percentages of suspended solids. In practice, chemicals

are assumed to be removed in proportion to the suspended solids. Where nutrients or bacteria are of concern, the type of SWMP or its characteristics may be explicitly specified (eg. wetland SWMPs and aquatic plantings in wet ponds for nutrient removal; extended release (72 hour) or U-V disinfection for bacteria control). These specifications are only tested at the watershed/subwatershed planning level in extreme cases because of the data requirements and uncertainty inherent in water quality modelling.

The effect of urban SWMPs on in-stream temperature and in some cases dissolved oxygen are modelled explicitly in watershed/subwatershed planning, if there is a significant concern for sensitive aquatic resources. Urban SWMPs may significantly affect these properties of the stream and must be considered, in conjunction with other management options (eg. revegetation of the riparian corridor, removal of on-line ponds, creation of riffle areas).

Ecological Assessments and Modelling

A primary focus of watershed/subwatershed planning is to maintain or improve the ecological health of the watershed as land use change occurs. As a result, it is necessary to evaluate the impacts to the physical systems and indicators (discussed above) in terms of their resulting impact on biological systems. The methods used to conduct this assessment range from the use of professional judgement and literature, to the use of empirical models.

In many watershed/subwatershed plans modelling tools are utilized to assess the impacts of alternative plans on biota. Some of these techniques (eg. Habitat Suitability Index (HSI) models) use input parameters such a stream temperature and flow depth, along with geomorphological and biological data to predict impacts. Where such models are used, the impact of urban SWMPs may be evaluated directly through consideration of their influence on stream temperature and flow.

Some ecological models used do not allow for the evaluation of urban SWMPs. These models address the impacts of urbanization based on empirical data and are most often used to indicate thresholds for the extent of intensive land use (both urban and agricultural) which may be allowed without significant ecological damage. The use of such models can indicate the limits to urban growth which should be considered.

1.8 Elements of the Watershed/Subwatershed Plan

The process of developing a watershed/subwatershed plan generates a great deal of information which must be documented and synthesized to formulate the final plan. The final crafting of the plan requires the input of all disciplines on the consulting team, as well as members of the Steering Committee and ultimately, the public.

The interim Provincial guideline "Subwatershed Planning" provides an outline of what should be included in a subwatershed plan. It is generally applicable to both watershed and

subwatershed plans, recognizing differences in the scale and scope of these plans. The outline is reproduced in the following section.

1.8.1 Contents of the Subwatershed Plan

Subwatershed plans will recommend how water resources and related resource features are protected and enhanced to coincide with existing and changing land uses. As well, other major uses of water, outside the municipal planning process, need to be factored into land use decisions. These uses include withdrawals, channel alterations, diversions, etc., that are carried out under various pieces of legislation and the federal Fisheries Act. **Briefly, subwatershed plans allow water-related environmental objectives and targets to be set at a time when they can be effectively incorporated into land use planning documents.**

Specifically, subwatershed plans will:

- **Identify the location, areal extent, present status, significance and sensitivity of the existing natural environment within the subwatershed.** A complete range of environmental features, and influences on natural systems must be addressed including the quantity and quality of surface water and ground water, aquatic and terrestrial habitat, fisheries and wildlife communities, soils and geomorphology, how they are linked and how these linkages are influenced by human activities.

- **Establish goals and objectives for management of the subwatershed.** Where a watershed plan exists, it will provide watershed goals and objectives that must be recognized in subwatershed plans. Where no watershed plan exists, local and downstream uses/needs, e.g., swimming, drinking water supplies, must be addressed in the subwatershed goals and objectives.

- **Identify environmentally sensitive or hazard lands, and recommend, with reasons, appropriate environmental management practices.**

- **Identify lands where development may be permitted, provided it is designed to ensure that ecological functions are protected and maintained.**

- **Provide directions for the screening and selection of Management Options for the subwatershed.** Recommended practices should address a range of activities including agricultural, development servicing, aggregate extraction, woodlot management, retrofitting activities, water taking, etc.

- **Address cumulative impacts of changes to subwatersheds on the natural environment**, and determine how existing and future land uses can compatibly exist with the natural environment.

- **Integrate disciplines, policies, mandates and requirements of all agencies and interests** in a subwatershed to resolve conflicting or changing approaches to watershed management.

- **Provide direction, consistency and uniformity of conditions of approval** for individual municipalities within the subwatershed.

- **Promote public participation in and support for** subwatershed planning.

- **Establish an implementation strategy** that identifies roles, responsibilities of all involved parties and timing of works and programs to ensure that chosen environmental and development practices are implemented.

- **Outline requirements for monitoring programs and information updates as well as facilities recommended by the plan.**

- **Provide technical information** that will assist in the development of Community Plans and the design of subdivisions.The subwatershed plan should be a readable, concise document that presents methodology, assumptions, findings and recommendations. These plans are intended for wide readership and use by resource managers, elected officials, landowners and developers. Technical studies essential to the development of the subwatershed plan should be provided in separate appendices. These appendices should provide all pertinent technical data and analyses in support of the recommendations of the subwatershed plan. Technical information in both the report and the appendices should be presented graphically wherever possible for easy interpretation.

Contents of a subwatershed plan are described in Table 1.1 The plans will provide a range of information and practical recommendations on boundaries, links to other planning/environmental tools, management objectives, and methods for implementation.

1.8.2 Key Direction Provided in the Plan for Urban Development

The watershed or subwatershed plan may contain substantial direction in terms of management programs and specific projects which are to be undertaken by different agencies or authorities. Since a specific implementation strategy will have been developed for these, the responsible agency or authority will be aware of what must be accomplished and where. The watershed or subwatershed plan however must also give direction on how to deal with growth in the future. In order to do this it must specify constraint areas, development criteria and land use limitations.

Constraint Areas

Constraint areas are those areas inside of which no development should be permitted. The identification and delineation of the constraint areas represents the primary protective focus of the watershed/subwatershed plan, but in certain cases the selection of constraint areas may

actually be an enhancement measure. The intention in selecting a system of constraint areas is to ensure the preservation and protection of the key physical and biological systems of the watershed/subwatershed, and to maintain the function that these key areas provide.

The basic framework of constraint areas is often known at the beginning of watershed/subwatershed plan development. Floodplains, designated wetlands and woodlots, municipally designated Environmentally Sensitive Areas (ESAs) or provincially designated Areas of Natural and Scientific Interest (ANSIs), are generally known and enjoy variable degrees of protection under existing provincial, conservation authority, or municipal policy. During the course of watershed/subwatershed plan preparation the basic framework of constraint areas may be expanded to include:

- protective buffers or setbacks (ie. from a water course or top of valley).
- areas which perform a significant natural function which is vital to the integrity of the watershed/subwatershed (such as recharge areas, head water source areas etc.).
- undesignated areas of significant natural value (such as the valleys of minor tributaries).
- areas which are necessary for the preservation of the value of other protected areas (eg. corridors or linkages between significant natural areas).

At a watershed planning level the direction provided for constraint areas will be generic or will point to the need for more detailed investigation at the subwatershed planning level. It is appropriate in a watershed plan to specify standard stream, wetland or woodlot buffers, and to identify areas of significant natural function, natural areas to be investigated further and potential linkages. All of these would be evaluated and assessed in more detail at the subwatershed plan level. At the subwatershed plan level, variable buffer widths which meet the needs of the particular biological systems may replace the generic buffers specified in the watershed plan. Detailed evaluations would be carried out for the new potential natural areas, areas of significant function and the potential linkages, in order to confirm or deny their value and importance.

The watershed or subwatershed plan will contain maps showing the boundaries of constraint areas. The boundaries will be consistent with the scale of the Plan and will not be of legal or property survey precision. The Plan should indicate the procedures for subsequent setting of actual boundaries and any studies that may be required prior to adjustment of the boundaries.

The network of constraint areas identified in watershed and subwatershed form the core of the areas which will be protected from encroachment. The lands which remain outside of the constraint areas are potentially developable (using mitigative measures such as SWMPs) within the limits which may be set to avoid unacceptable cumulative impacts.

Plans for urban development should have regard for the boundaries of constraint areas. In general, development will not be permitted within the specified boundaries, although in some cases, uses such as stormwater management facilities may be permitted.

Cumulative Impact and Land Use Restrictions

In most watersheds or subwatersheds there is a limit to which urban development and growth can proceed without causing irreparable damage to the natural systems which support the watershed ecosystem. The threshold at which such damage may occur is variable and largely depends on the physical and biological characteristics of the area. The identification and protection of constraint areas and the mitigative measures specified as part of the criteria to govern development, are methods of extending the development threshold by maximizing the protection provided to the key elements of the watershed. In some instances however, there may be a need to further limit the cumulative impact of urban development by restricting the levels of imperviousness that are allowable.

In such cases there are a variety of options ranging from the control of development form (eg. higher density, clustered and buffered development) to complete restriction of portions of the watershed/subwatershed to uses which do not produce a significant change in the hydrologic or hydrogeologic regimes of the watershed. The selection of the appropriate approach is not solely a watershed/subwatershed planning concern, but rather may require an in-depth assessment of land use needs of the community. It is incumbent upon the watershed/subwatershed plan to identify the concerns for cumulative impact, to identify the areas which could be subject to land use restrictions and to identify the levels of imperviousness which would be acceptable to avoid undesirable impacts. The watershed/subwatershed plan is not intended to set land use policy but it is required to set the environmental factors which must be met by future land use decisions.

Development Criteria

The watershed/subwatershed will specify development criteria which will apply to all lands where development may take place. Development criteria specified in a watershed plan will normally take two forms. The plan may require that prior to development, specific studies be undertaken. Examples of studies that may be required include:

- flood plain mapping of unmapped tributaries.
- establishment of the specific location of buffers or setbacks.
- completion of an erosion control plan (during construction).
- detailed evaluation of potentially important natural areas, corridors or linkages (identified in the watershed/subwatershed plan).
- selection of specific locations for rehabilitation or enhancement measures (if these have been specified as necessary compensatory actions associated with development, in the watershed/subwatershed plan).
- studies which may be needed to confirm the feasibility of management options specified in the watershed/subwatershed plan (eg. infiltration capacity).

Table 1.1
Components of a Subwatershed Plan

A Subwatershed Plan clearly presents the following information:

- **SUBWATERSHED BOUNDARIES** including rationale for their establishment.

- **RELATIONSHIP OF SUBWATERSHED PLAN** to watershed plans (if available), and to other urban drainage, environmental, land use and planning studies and programs.

- **IDENTIFICATION OF FORM AND FUNCTION OF NATURAL SYSTEMS** including land uses, natural features, linkages, and surface and ground water systems. Identification of existing systems should include aquatic and terrestrial features/habitats, and the quantity and quality of surface and ground water resources, relationships and water-related dependencies, and factors influencing the viability of the resources.

- **SUBWATERSHED OBJECTIVES** for public health, public safety, aquatic life, resource management, floodplain management, and urban, agricultural and other land uses.

- **PLAN RECOMMENDATIONS**

  - specify areas for protection, rehabilitation, and/or enhancement. It should be clearly noted where changes within the subwatershed should not occur, along with appropriate setbacks from natural areas, and recommended management strategies for these areas

  - establish areas that can be developed in a manner compatible with subwatershed objectives; identify how this can be achieved through use of best management practices and drainage system design that will protect, enhance and/or rehabilitate natural areas and systems

- **IMPLEMENTATION PLAN** outlining:

  - policy/guidelines to direct development planning and design
  - design, function, siting and timing of facilities
  - funding of works, inter-agency review/approvals, and regulation requirements
  - recommendations and responsibilities for future studies
  - operation and maintenance responsibilities
  - monitoring program and responsibilities
  - approaches and responsibilities for information updating and corrective actions
  - time frame for review/update of plan

Source: Table 1 from "Subwatershed Planning" 1993

- additional "subwatershed" or tributary studies in areas where there was insufficient planning information to specify locations and types of urban SWMPs.

In addition to the detailed studies which may be required, the watershed/subwatershed plan will normally specify the performance criteria to be used in developing stormwater management plans. The performance criteria should include:

1) The need for lot level controls and site planning techniques to promote infiltration and maintain the water balance. This may be a qualitative directive (where baseflow is not a primary concern) or it may quantitative if groundwater modelling has been conducted. If a specific numerical target is set, it should indicate the target and the assumed percolation rate used in the subwatershed plan and should make provision for adjustment (increase or decrease) of the infiltration target according to the soils encountered on a site basis.

2) Approximate location and allowable types of end-of-pipe SWM facilities

3) Required levels of control (storage or recharge volumes and retention times) for water quality, erosion protection and flood control.

4) Special design requirements for SWMPs (eg. aquatic plantings for enhanced nutrient control, thermal mitigation measures, spill capture/control enhancements etc.)

5) Requirements for special purpose SWMPs (eg. oil/grit separators for specified uses, U-V disinfection).

1.8.3 Plan Finalization

The final steps in both watershed and subwatershed planning studies involve plan finalization, presentation of the plan and review and adoption by all agencies and the public. Important elements in finalizing the plan include the development of an implementation plan and a monitoring plan. The interim Provincial Guideline "Water Management on a Watershed Basis : Implementing an Ecosystem Approach" provides guidance on this aspect and is reproduced in the following sections.

Implementation Roles and Responsibilities

The scheduled events and responsibilities for implementing the recommended actions are a delivery mechanism that should provide answers to the questions:

- **what** doable tasks are needed to accomplish each recommended action ?

- **who** is accountable for each task ?

- by **when** is each task to be accomplished ?

- **how** will monitoring results be used to modify implementation ?

Implementation of recommended actions is likely to take place largely through land use planning decisions, but others will be the responsibility of participating agencies, through such things as approval processes, regulations and permits. If there has been consistent interaction among participating agencies throughout the plan development process, it is likely that by the implementation stage, all participants will know what they are required to do.

The issues and recommended actions in watershed plans involve the jurisdictions and mandates of a range of agencies, including municipalities, conservation authorities, provincial ministries, First Nations and private interests. All participants can effectively use existing mechanisms and tools, like legislation, policies, procedures and approval processes, to implement the watershed plan. Provincial agencies such as MOEE, MNR, MMA, and OMAF have a number of key pieces of legislation that can be used to carry out recommended actions. These include MNR's *Lakes and Rivers Improvement Act, Fisheries Act, Endangered Species Act, Trees Act,* and *Provincial Parks Act.* Also useful are MOEE's *Environmental Protection Act, Environmental Assessment Act,* and *Ontario Water Resources Act,* as well as OMAF's *Topsoil Preservation Act.* A listing of provincial legislation is available in Ministerial Responsibility for Acts, Ministry of Government Services, Queen's Printer for Ontario, 1991.

Conservation authorities are encouraged to administer the provisions of the *Conservation Authorities Act,* and Fill, Construction and Alteration to Waterways regulations pursuant to Section 28 of the Act. Municipalities are encouraged to administer the provisions of the *Municipal Act* and the *Planning Act* and plans and by-laws adopted according to these acts.

Conservation authorities, where they exist, are encouraged to coordinate watershed management, and can play a key role in plan implementation in the following ways:

- Assist municipalities and planning boards to incorporate the intent and recommendations of the watershed plan into the land use planning process and appropriate planning documents.

- Review and comment on proposed planning that may have implications for the watershed plan or water management.

- Make representation or provide technical expertise to the Ontario Municipal Board or other appeal bodies, where a matter related to the watershed plan and water management may be an issue.

- Consult with ministries, public agencies, boards, authorities and municipalities on matters pertaining to the watershed plan and water management, as appropriate.

- Inform the general public about the principles and practices of watershed management, and provide information on the characteristics and consequences of various land use and development activities.

Where conservation authorities do not exist, the Ministry of Natural Resources and the Ministry of Environment and Energy are responsible for coordinating a program to address watershed planning and management.

Monitoring - Auditing the Success of Watershed Management

The relative success of watershed management decisions or actions should be audited using monitoring. Implementation of the plan should be a flexible and iterative process which both directs and responds to status changes in the adherence to recommendations and the achievement of the plan's goals. A monitoring program can identify the environmental conditions that indicate progress. There are two major components to monitoring: monitoring the success of the plan, achievement of its goals and objectives (response of the system to the implemented plan); and monitoring the performance and success of the tools used to achieve the objectives developed by the plan.

Implementing the watershed management plan will require monitoring data for a variety of uses. It is important to remember that **monitoring programs need not all be sophisticated or highly technical.** Sometimes, observation will suffice. Local citizens can be enlisted to watch for and report the status of or changes in environmental conditions. This will provide the public with a tangible opportunity to participate in achieving the watershed plan's ecosystem objectives, and thereby the integrity of their own surrounding environment. It will also probably reinforce and maintain interest in the plan's success in achieving its management goals. Another method is to identify appropriate "indicator species" for ecologic integrity, and establish "water budgets" for aquifers.

As well, it is important to note that **monitoring need only be applied to issues or conditions in the watershed that the plan has identified.** Furthermore, the plan can even identify some aspects to be monitored by federal or provincial agencies, as aspects to be incorporated into their ongoing state-of-the-environment monitoring programs.

If monitoring reveals successful initiatives, these should be documented and shared with agencies that might benefit from this knowledge.

Currency - Keeping the Watershed Management Plan Up-to-date

Effective watershed management is an iterative process, taking full advantage of both the successes and mistakes of implementation. Lessons learned from performance monitoring during implementation should be used to make appropriate revisions in watershed management programs.

As a general rule, it is appropriate to re-evaluate a watershed plan when land use changes are identified in an official plan of a municipality in the watershed.

1.9 Public Involvement and Consultation

As indicated in the draft provincial guidelines on watershed and subwatershed planning, public involvement and consultation are fundamental to the water management planning processes. Watershed and subwatershed plans are not subject to the requirements of the Environmental Assessment Act, but many of the specific projects which may be implemented in the execution of the plan, may be subject to municipal or conservation authority class environmental assessments. As such, it is common to conduct watershed and subwatershed planning in the spirit of the environmental assessment process, which includes among other things, the implementation of a public notification, information and consultation program. By conducting the watershed/subwatershed plan in the spirit of the environmental assessment process, it is often possible to utilize the work completed in the watershed/subwatershed plan to fulfil the requirements of the early stages of subsequent class environmental assessments.

In practice, public involvement and consultation may take many forms through the course of the studies leading to plan development. Naturalist groups, rate payers associations and individuals can be sources of useful information ranging from historic practices to community priorities. Many contacts are made with the public through the work completed for the background review and subsequent field studies.

Formal notification of the general public is normally undertaken at the commencement of the watershed/subwatershed plan. In most cases notification is done through newspaper advertisements. This notification may be supplemented by direct mailings to local environmental and rate payers groups (where the municipality and/or conservation authority maintains such a mailing list). The purpose of this preliminary notification is primarily to make members of the public aware of the planning which is to be done and to provide them with an opportunity to be placed on the project mailing list, so that they can become part of the planning process.

Public meetings and/or information centres are usually conducted at three points in the watershed planning process:

- at the end of the background review phase, prior to finalization of the detailed work plan (the meeting usually also addresses the plan goals and objectives).
- prior to commencement of detailed evaluations and assessments (in order to discuss potential management options and assessment methodologies).
- at the time the recommended plan is first available, prior to agency or municipal adoption.

The selection of the points for public meetings is based on providing the public with the opportunity for input at times where they may influence the outcome or directions taken in planning. The points identified above will allow the public to influence:

- the emphasis to be undertaken in the detailed studies (through input on goals, uses and provision of information).
- the range of management options to be considered and the means of assessing these.
- the final overall strategy to be adopted in the watershed plan.

Care must be taken in conducting public consultation to ensure that the water management and environmental protection focus of the plan is made clear. Purely land use planning issues will inevitably tend to impinge on the public consultation process. All possible efforts should be made to ensure that this tendency does not limit discussion or input, or dominate the environmentally related issues.

2.0 SUBDIVISION / SITE PLANNING

2.1 Introduction

This section of the manual provides a proposed subdivision/site planning methodology for development which promotes water management and is environmentally sustainable. Whenever subdivision/site planning is referenced in this section of the manual, the term applies to subdivision planning, site planning and engineering, landscape design, architectural and building design, as well as local street design. It is important that subdivision/site planning and the design of urban SWMPs are undertaken in an integrated process to ensure that an environmentally responsible planning process is implemented.

It should be recognized at the outset that there is no legislative authority to plan subdivisions/sites using the methodology which is outlined in this chapter. In recognition of the ecosystem approach taken in this manual, however, this chapter has been included as an integral part of the environmental-land use planning process. Historically, environmentally integrated subdivision/site planning techniques have not been implemented on a widespread basis in Ontario. The concept of integrated subdivision/site planning, however, is an integral link between stormwater management at a site level and subwatershed planning.

It has been mentioned throughout the manual that watershed/subwatershed planning and stormwater management are evolving. Subdivision/site planning is seen as a necessary part of this evolution. As such, the following subdivision/site planning guidelines have been included in the manual as "forward thinking" Stormwater Management Practices to promote the education and awareness in this important aspect of development.

2.2 Subdivision/Site Planning and Stormwater Management Practices

It is important to understand that subdivision/site planning is a fundamental determinant of the overall change in the hydrologic cycle for a given development. The way a development is planned, and the specific design criteria adopted by the planner/engineer/designer, can have a great impact on the level of success achieved by the stormwater management measures which are implemented. However, the significance of subdivision/site planning is not always well understood by the landowners, their consultants, local politicians or the public. The following discussions provide an appropriate framework to understand this important aspect of the development process.

- ***Watershed and subwatershed planning: environmentally responsible land use policies must be supported by environmentally responsible site design.***

  The preparation of watershed and subwatershed plans is recognized as an essential part

of the land use planning process. The watershed and subwatershed planning process is integrated with the official plan preparation and review process to ensure that an ecosystem approach is adopted in making land use planning decisions.

Watershed and the subwatershed plans address the ecosystem at a regional level. At this level, land use decisions are made as generalized policies and guidelines, and environmental information is often collected and interpreted at a broad scale. While these broad scale evaluations allow the development of strategies which are not possible through site specific evaluations, it is not always possible to interpret the merits or demerits of various individual development proposals at this stage.

The fundamental objectives of watershed and subwatershed planning can only be realized if the principles of watershed/subwatershed planning are also applied during the planning and design of individual development projects. At this point of the development process, detailed site information is available and the physical parameters of the proposed development are determined. The subdivision/site planning stage is therefore an important step in the planning process when the impact of the development proposal on the environment can be specifically assessed. The integration of land use planning and environmental planning at a regional or district level must be extended to the process of site development and design.

- ***Good planning integrates the design of a site and the design of the stormwater management facilities into one process.***

Historically, the preparation of subdivision plans, site development plans as well as building and architectural design plans has not involved early input from environmental planners, hydrogeologists, ecologists and water resources engineers. The landowners and the planners/designers prepare the plan based on the performance standards set by the municipal by-laws or guidelines (such as setback, floor space index, density, height, etc), and the business objectives set by the landowners (such as total leasable floor area to be achieved, number of units for sale and the number of parking spaces to be provided).

Water resources engineers, and other associated professionals, are typically employed to address stormwater management after a preliminary site plan has been prepared. This process has inevitably made the proposed stormwater management facilities 'remedial' in their nature since they are designed to handle a predetermined amount of runoff and to mitigate the negative impact of the proposed development. An alternative approach is advocated: that the objective to reduce the root causes of negative impact on water management must be adopted as one of the basic design criteria **directing** the preparation of the site plan. The important aspect of good subdivision/site planning is that it should aim at reducing or preventing adverse impacts instead of mitigating them.

- *Public perception and implementation of innovative subdivision/site planning approaches.*

  There is a perceived public attitude that many of the proposed environmentally friendly subdivision/site planning criteria such as cluster housing forms, roadside ditches and the inclusion of runoff infiltration devices within the residential lots are undesirable and represent a reduction in the level of service. This perception extends to some municipalities whose development standards may constrain the use of subdivision/site planning techniques. As a result, developers may hesitate to include these design criteria in their site development plans. Nevertheless, the attitude of the public is changing as more and more innovative projects are delivered into the market and the public sees the value of these new design concepts.

  It is important to recognize that with increased implementation of innovative subdivision/site planning techniques, and with more projects designed based on environmentally responsible development planning criteria, public perception will evolve and values will change. Creative stormwater management design ideas should be encouraged and adopted as part of the design during the subdivision/site planning stage of the development process.

- *The most environmentally sound design is generally the most economical*

  Subdivision/site planning generally reduces the cost of the development due to :
  - lower grading requirements/costs
  - less tree clearing costs
  - lower servicing costs (swales instead of storm sewers)
  - lots with mature trees are more saleable/valuable
  - lots that back on to greenbelts are more saleable/valuable
  - tourism dollars in areas with sports fishery
  - lower end of system clean up costs (ie. dredging, etc.)

2.3 Subdivision/Site Planning and Design Objectives

There are many excellent references, such as "*Protecting Water Quality in Urban Areas - Best Management Practices for Minnesota*" (published by the Minnesota Pollution Control Agency, October 1989) which illustrate the value of subdivision/site planning. These references were reviewed in the formalization of the objectives to direct the preparation of development plans. These objectives are shown in Figure 2.1 and can be listed as follows :

- Reproduce the pre-development hydrological conditions

- Confine development and construction activities to the least critical areas

- Maintain the overall desired density of development by allocating higher densities to areas suitable for development (if required)

- Minimize changes to the existing topography

- Preserve and utilize the natural drainage system

Whenever a design decision is made during the subdivision/site planning stage of a project, it should be assessed against the above-noted objectives.

These objectives must direct the thinking of the planners or the designers in the choice of site design approaches, site configuration, building location and orientation, massing, site grading, circulation and access, as well as landscape design.

To assist site designers, the objectives have been translated into a subdivision/site planning methodology (Section 2.4) which should be used to prepare a development layout.

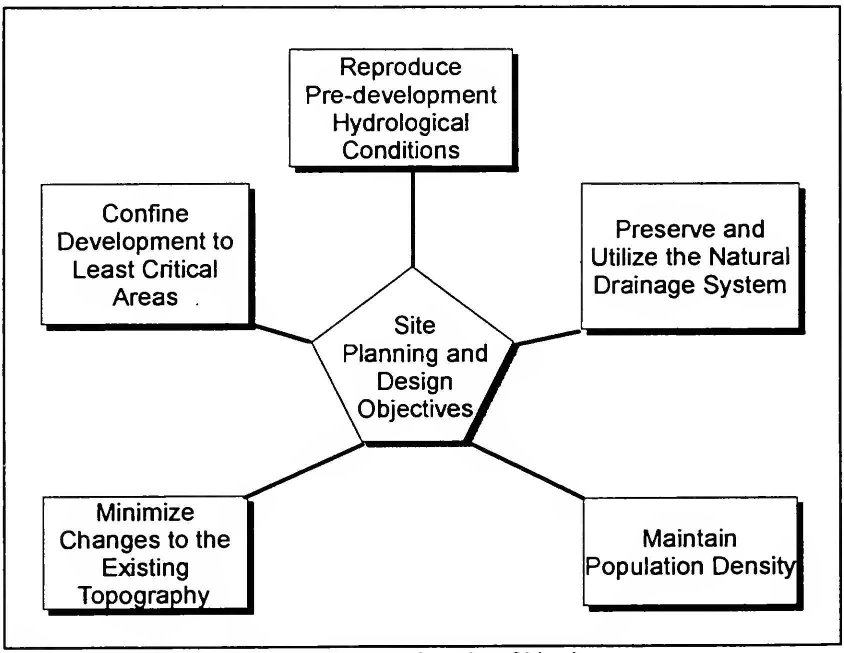


Figure 2.1 Subdivision/Site Planning and Design Objectives

2.4 Subdivision/Site Planning Methodology

As with most environmental undertakings, the main determinant of good subdivision/site planning is the process or methodology used to layout a development. Accordingly, a systematic process of site planning is proposed. This process can be summarized as follows (Figure 2.2):

1. **Agency Consultation** - identify existing resource mapping/data and natural resource concerns

2. **Resource mapping** - identify significant natural functional areas for protection

3. **Designation of development area** - determine the areas for development based on the resource mapping information

4. **Evaluate stormwater management requirements** based on the preliminary site plans. Indicate locations and land area required to be formalized into the site plan for the purposes of stormwater management in the different plans

5. **Adoption of environmentally responsible site planning and design criteria** - apply a set of environmentally responsible design criteria to the development area during the preparation of the site plan options

6. **Finalization of the subdivision / site layout** - examine the various site plan options based on the criteria and select the option that best meets the site planning and design objectives

This methodology is presented in detail below:

2.4.1 Agency Consultation

The regulatory agencies (Ministry of Natural Resources, Conservation Authority, Local Municipality, Ministry of the Environment and Energy) should be contacted for information on existing areas which are deemed to be environmentally significant.

2.4.2 Resource Mapping

Resource mapping is required to ensure that significant natural resources are maintained or enhanced. On an appropriate scale ($\leq$ 1:2000) map of the proposed development site an outline of the following resources should be clearly delineated :

- ESA/ANSI areas
- watercourses, kettle lakes and other waterbodies

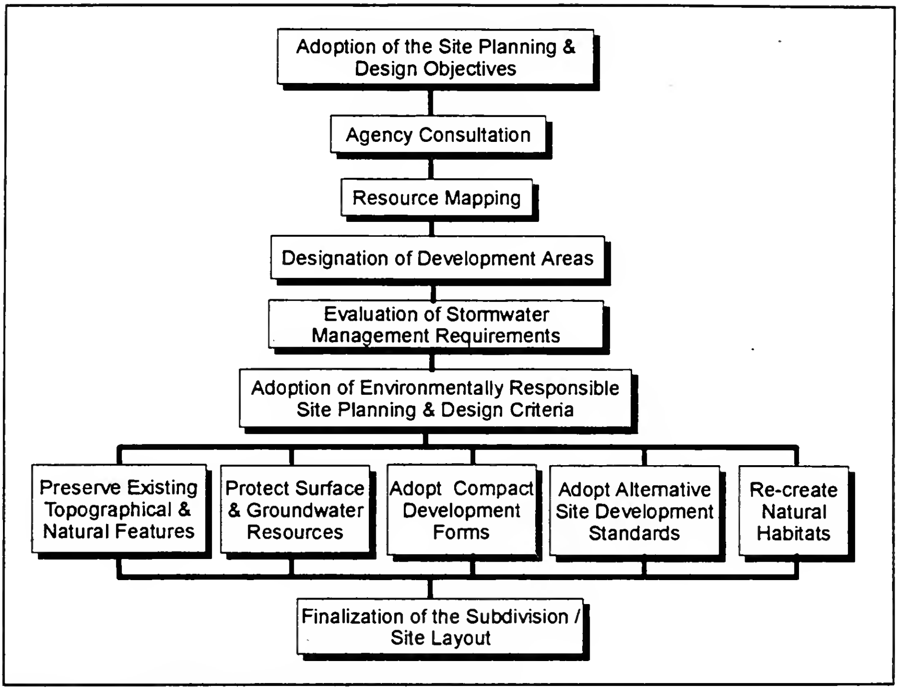


Figure 2.2 Subdivision/Site Planning Methodology

- wetlands
- significant vegetation/woodlots
- wildlife corridors
- high recharge potential areas
- regulatory floodlines and/or fill lines
- stream and valley corridors
- bank instability and/or erosion setbacks
- steep sloped areas

Much of the information required for resource mapping may have been delineated (usually at a larger scale) in the watershed or subwatershed plan (if it has been completed). Reference should be made to these plans as part of the site investigations.

ESA/ANSI Areas

The Ministry of Natural Resources and the Conservation Authority should be contacted

for mapping which indicates ESA and ANSI areas. Municipalities should also be contacted for mapping related to any locally significant areas (LSA). These areas should be transferred to the site mapping and clearly shown on development submissions.

Watercourses, Kettle Lakes and other Waterbodies

Watercourses, lakes, and other waterbodies should be denoted on the resource mapping. Ontario Base Mapping (1:2000, 1:10000), where available, is a useful source of information which will indicate surface water resources.

Larger scale topographical mapping will also indicate most surface water resources. In some cases, however, all surface water resources may not be delineated on these larger scale maps in order to preserve clarity (ie. in areas with high topographical relief - many contours).

In all instances, a site visit should be undertaken to confirm the surface water resources in the vicinity of the proposed development.

Wetlands

Provincially significant wetlands should be shown on the resource mapping. The class of wetland should also be displayed to indicate the importance of each feature. An environmental impact study (EIS) will be required if development encroaches within 120 m of a Class 1, 2, or 3 wetland boundary. (Although the Provincial Policy states that an EIS is required for Class 1, 2, and 3 wetlands, it is commonly requested for all classes of wetlands (1-7) in the field.) This study will determine the impacts from development on the wetland and the appropriate buffer width or mitigative measures. Wetland areas that are not provincially classified should also be shown on the mapping to help identify the hydrologic characteristics of the site in question. This will be useful to identify problem areas for development that should be avoided (ie. low wet areas) and constraints for the siting and implementation of stormwater management techniques.

Areas of Significant Vegetation

A terrestrial biologist should walk the site to identify the areas of the site with significant vegetation. Significant vegetation includes provincially significant, regionally significant, and locally significant communities. An area can also be deemed significant, in terms of its vegetation, if it provides a corridor or refuge area for wildlife, a food source for terrestrial/aquatic species, a significant hydrological function, and/or a buffering capacity to mitigate the effects of urban development on the stream and valley corridor system.

In some cases, information on the vegetation of a site can be obtained from the Conservation Authority, Ministry of Natural Resources, and/or local naturalist groups. A site walk/inventory is preferable, however, noting that most mapping/information will not be up-to-date and that values with respect to the importance of site vegetation have evolved dramatically in recent years (ie. what is important today may not have been regarded as important a decade ago). The limit of development should be the drip line of the vegetation. No earthworks should be permitted within 3 to 5 metres of the vegetation drip line to protect the vegetation's root system.

Wildlife Corridors

The significance of wildlife corridors is best addressed at the Subwatershed Plan stage or Watershed Plan stage. These plans should be reviewed if they exist. If a watershed plan and/or subwatershed plan has not been completed the Ministry of Natural Resources should be consulted for input. A site walk by a terrestrial biologist should be undertaken to confirm the recommendations of the watershed/subwatershed plan and the information provided by the Ministry of Natural Resources. Information from the site walk should be compared to the greenspace areas in the surrounding geographic area to determine if a wildlife corridor exists on the site. Significant wildlife corridors should be drawn on the resource map.

Recharge Areas

Boreholes and test pits are required to determine the groundwater recharge potential for the site. This investigation must be undertaken by a qualified soils consultant or geotechnical engineer. Information which needs to be collected includes soil types, soil depths, the depth to the water table, the degree of soil compaction, soil percolation rates, the estimated high seasonal water table depth, the depth to bedrock, and soil particle size distributions.

Soils with a percolation rate greater than 50 mm/h should be identified as recharge areas (ie. any development must ensure that recharge is maintained), and soils with a percolation rate greater than 100 mm/h should be identified as critical recharge areas (ie. areas that may be non-developable or require significant geotechnical investigation in support of development), given that the depth to bedrock and depth to the water table is greater than 3 m below the ground surface. The theoretical percolation rate (Table 3.1) based on the particle size distribution and degree of compaction should be compared to the measured percolation rate to ensure that the measured rate is reasonable.

Regulatory Floodline and Fill Line

The regulatory floodline and/or fill line should be shown on the resource map if the proposed development is adjacent to a watercourse. If a floodline or fill line has not been delineated, and is not required to be delineated (ie. upstream drainage area is small (< 125 ha) and the Conservation Authority is not concerned with flooding) it does not need to be shown on the map of the site. In cases where the floodline or fill line is not shown, the watercourse should still be shown as they may serve an important ecological function.

Stream and Valley Corridors

The area required to protect stream and valley corridors is best decided at the subwatershed plan level. In the absence of subwatershed planning, Section 3.4.8 provides guidelines for the establishment of buffer widths for stream and valley corridors. The stream and valley corridor area should be shown on the resource map.

Bank Instability and/or Erosion Areas

Areas susceptible to bank instability and erosion should be identified on the resource map. These areas will typically be within the stream and valley corridors. Table 3.4 provides guidance on the determination of area to be shown on the site map related to bank instability/erosion.

Steep Sloped Areas

Areas with a slope of greater than 20% should be identified on the resource map. These areas may be difficult to develop (ie. result in significant alteration to the natural topography) and should be noted as constraint areas.

The resource mapping information should be compiled into overlays of information sheets and maps for easy cross referencing. These overlays will illustrate the inter-relationship between the different elements of the ecosystem. At this stage, the planner/designer should start to determine where development should occur within the site to produce the least impact on the ecosystem.

A joint site visit (top-of-bank survey) can be conducted by the agencies and the proponent together to determine the limits of development and confirm the resource mapping. Although a top-of-bank survey will be beneficial in identifying significant natural resources to be protected, the regulatory floodline or fill line may take precedent over the top-of-bank survey, if the development limits established in the survey are within the floodline/fill line.

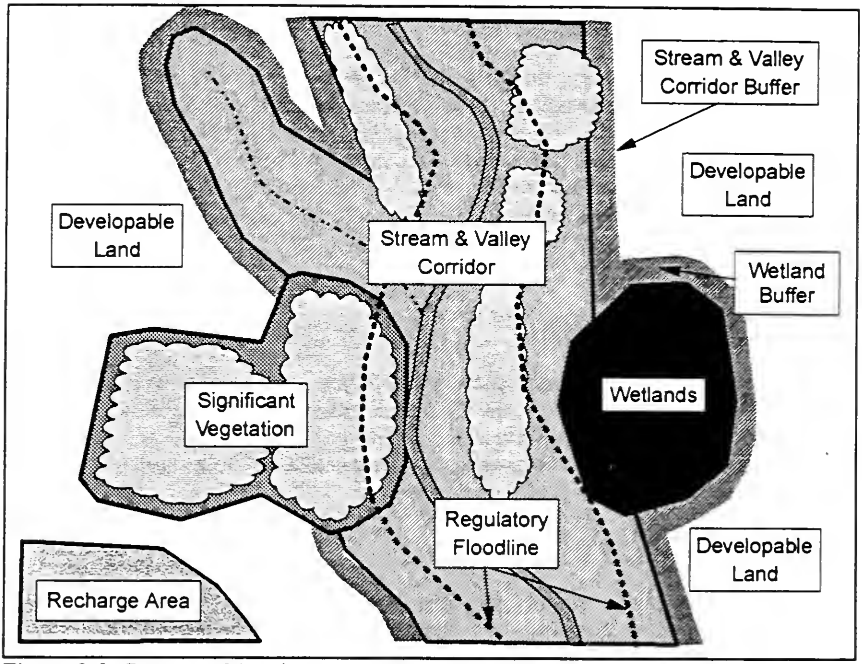


Figure 2.3 Resource Mapping

2.4.3 Designation of Development Area

Once the resource mapping has been completed, the remaining lands which are not identified as resource areas can be designated as development areas. They define the physical locations of possible development within the site that will have the least impact on the environment. Figure 2.3 illustrates the concept of resource mapping to determine developable land.

Once the area of developable land has been identified, a development layout should be prepared based on a set of environmentally responsible subdivision/site planning and design criteria.

2.4.4 Reserve Appropriate Areas for Stormwater Management

Subdivision/site planning must reflect the need for stormwater management. This requires interaction between planners/designers and stormwater management professionals to ensure that

there is adequate land area in appropriate locations designated for the purpose of stormwater management. The delineation of land area requirements for stormwater management will depend on the water management criteria which have been established for the site, the stormwater management measures that are contemplated, and the actual site planning that is proposed. The full range of stormwater management measures (lot level, conveyance, end-of-pipe) should be contemplated.

At this stage, a preliminary assessment of the required area/volume to properly treat the water, and the desired location of any stormwater management controls would be appropriate. Chapter 6 (Section 6.6) provides equations to estimate the land area required by different end-of-pipe SWM facilities.

Urban stormwater management practices should be located outside of the floodplain wherever possible. In some site specific instances SWMPs may be allowed in the floodplain if there is sufficient technical or economic justification and given that they meet certain requirements :

- The cumulative effects resulting from changes in floodplain storage, and balancing cut and fill, do not adversely impact existing or future development
- Effects on corridor requirements and functional valleyland values must be assessed. SWMPs would not be allowed in the floodplain if detrimental impacts could occur to the valleyland values or corridor processes.
- The SWMPs must not affect the fluvial processes in the floodplain.
- The outlet invert elevation from any SWMP should be higher than the 2 year floodline and the overflow elevation must be above the 25 year floodline.
- An online stormwater quantity facility would be acceptable if designed such that the bank full flows, and hence fish movement, are not impeded/obstructed, and that the foregoing requirements noted above are met. An online facility could only be proposed in the context of a subwatershed plan.

The location of end-of-pipe stormwater management facilities is a contentious issue since the use of tableland reduces the overall developable area. In an effort to minimize the loss of developable land municipalities should allow the use of park land dedication for SWMPs which offer passive recreational opportunities and follow the municipality's greenland strategies (parkland objectives) wherever possible.

2.4.5 Adoption of Environmentally Responsible Subdivision/Site Planning and Design Criteria

The following specific planning and design criteria are recommended:

- Preserve existing topographical and natural features

- Protect surface water and groundwater resources (stormwater management)

- Adopt compact development forms
- Adopt alternative site development standards
- Re-create natural habitat within the development areas

These criteria are discussed in the following sections.

<u>Preserve Existing Topography and Natural Features</u>

In order to preserve the existing topography and natural drainage system, buildings and roads should be located along high points and on flat slopes (Figure 2.4). Natural drainage swales should be used to convey runoff from the development to the receiving waters (Figure 2.5). This approach will reduce the area disturbed by cutting and filling along the slope and minimize the amount of surface area susceptible to erosion.

The application of this criterion must be made with consideration for the visual impact of locating buildings on and along the ridgelines of the landscape. To avoid the visual intrusion of buildings along attractive natural ridgelines and the disruption of existing prominent landforms, it may be necessary to site the buildings and the access roads along the contouring slopes.

<u>Protect Surface Water and Groundwater Resources</u>

The site plan should adopt site specific stormwater management approaches to reflect local soil conditions to protect the quantity and quality of the groundwater and the hydrologic regime in the surface waters. Stormwater is not 'wastewater' that should be disposed of as soon as possible. Instead of transporting water away as soon as possible without any regard for the potential adverse downstream impact, stormwater lot level and conveyance controls should be considered to mitigate the increase in surface runoff which occurs as a result of development, and to promote infiltration. These techniques are discussed in detail in Chapter 3 of this manual.

<u>Adopt Compact Built Forms</u>

In areas where development is restricted due to environmental features there may be a reduction in population density due to a reduction in developable land.

In order to develop with the same population density given a reduction in developable land area, compact development forms can be investigated. To accomplish this, compact

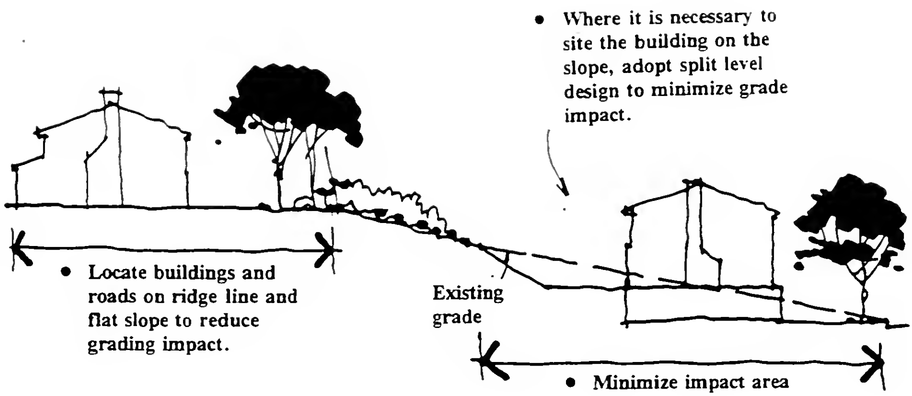


Figure 2.4 Preservation of Existing Topography

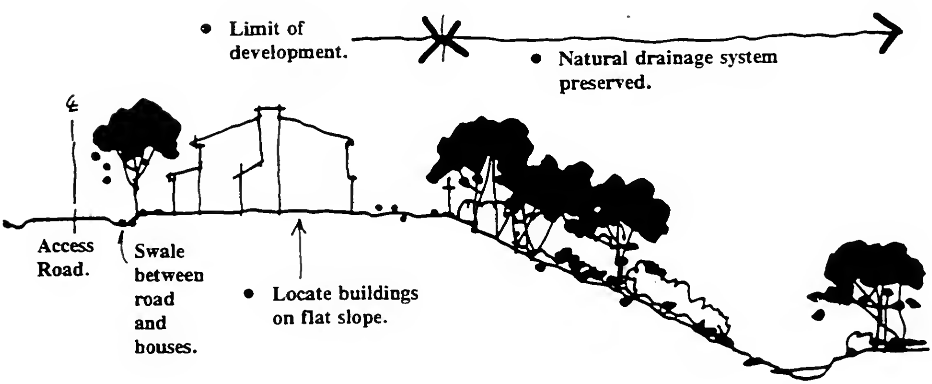


Figure 2.5 Preservation & Utilization of the Natural Drainage System

housing forms such as cluster single dwellings, medium density townhouses and low-rise apartments, and high-rise apartments can be adopted. This will allow the site to achieve a certain level of development density and minimize the gross area of the building coverage at the same time. This will, in turn, reduce the extent of disturbance to the site and the amount of site works required. Figure 2.6 illustrates the concept of maintaining density with single detached cluster housing while reducing the overall development area. The feasibility of single detached cluster housing is dependent on the use of alternative development standards.

The Ministry of Housing and Ministry of Municipal Affairs are actively promoting compact, higher density housing forms in the Province of Ontario. Compact, higher density housing forms are presented in Table 2.1 and shown in Figures 2.6 and 2.7.

Table 2.1 Compact Housing Forms
Cluster single lots with reduced lot frontages and alternative road/grading standards
Higher density forms such as duplex and semi-detached
Condominium singles
Medium density housing forms such as townhouses, fourplex and low-rise apartments
High density housing such as high-rise apartments

Adopt Alternative Site Development Standards

Many of the compact development forms recommended above can only be implemented with flexible site design standards (building setbacks, grading requirements, minimum street gradient and turning radius, width of internal streets, locations of site services, provision of street boulevard areas).

Alternative development standards are generally allowed in non-freehold development projects (ie. projects in which the services (roads, stormwater management facilities, etc.) are not municipally maintained - such as condominiums). Any public rights-of-way, public areas, and freehold residential lots, however, have to comply with the normal municipal planning and engineering (grading, servicing) standards. Public streets are designed to have a wide right-of-way and gentle gradients. These standards may limit the implementation of alternative housing forms to non-freehold developments. The adoption of alternative cluster single lots for the typical freehold development, for

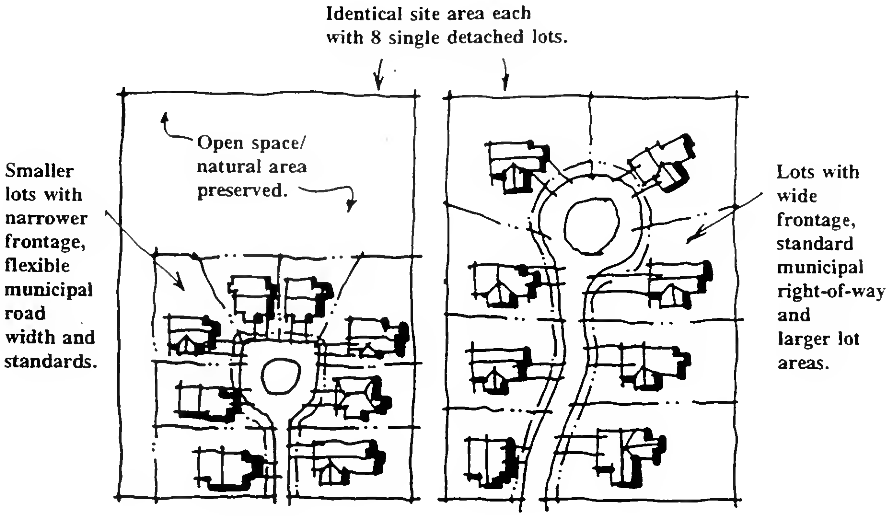


Figure 2.6 Cluster Single Detached Dwellings

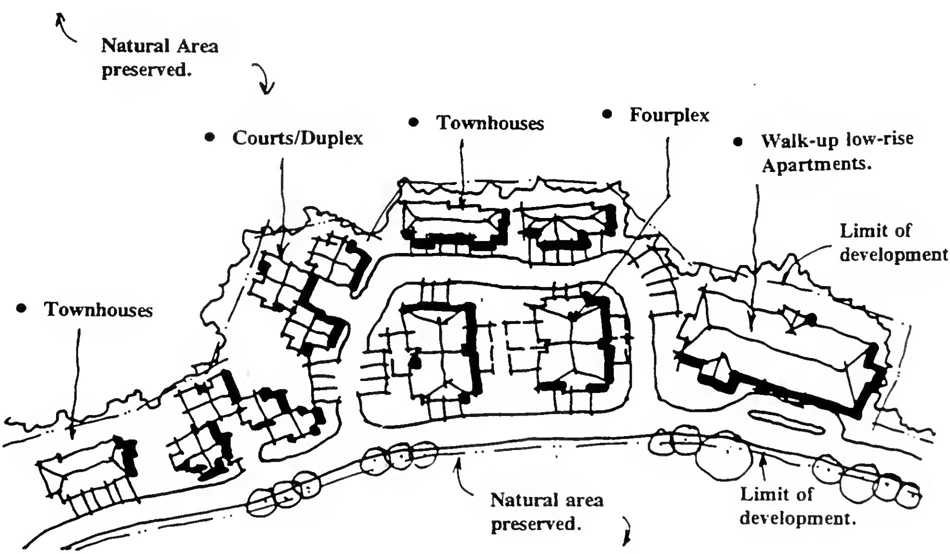


Figure 2.7 Other Forms of Cluster Housing

Alternative development standards complement reduced lot frontages and depths to reduce the overall development footprint. The Ministry of Housing has recently completed coordinating the production of a guideline entitled : "Making Choices: Alternative Development Standards Guidelines". The guideline reviews current municipal standards and recommended alternative standards to reduce development costs, promote compact urban form, and mitigate environmental impacts.

Some alternative engineering standards which help to reduce the overall footprint of development include :

- reduced road widths on local roads

  Reducing the road width to 6 m on local roads allows for two way traffic without street parking or one way traffic with parking. This reduces the overall pavement area, and hence costs, for the subdivision. The reduction in the pavement area will minimize the amount of land to be disturbed and grading works. It will also provide more flexibility for the planner/designer to align the proposed road along existing contours and integrate it into the existing landform.

- reduced cul-de-sac turning radius

  A reduction in pavement and overall land consumption can be achieved if the cul-de-sac turning radius is reduced from 14 to 11 metres.

Other alternative engineering standards which minimize environmental degradation and changes to the natural function of the land are shown in Figure 2.8 and include :

- a wider range in allowable lot grading

  A reduction in the minimum allowable lot grade promotes natural infiltration and inadvertently creates greater depression storage. Due to the problems of physically being able to grade below 2%, there should be an elevated apron around buildings (within 2 to 4 metres) to ensure that water does not drain towards the building foundation.

  Flatter lot grading should be promoted in naturally flat areas but radical changes to the existing topography should not be made to provide flatter lot grading. Municipal grading standards may also need to be modified for development within areas of varying topography to permit steeper lot grading. This flexibility will assist the designer to site the buildings along the slope and fit the built form into the terrain with minimum disturbance to the existing topography.

- higher maximum allowable slopes on roads (10% instead of 6%) and individual

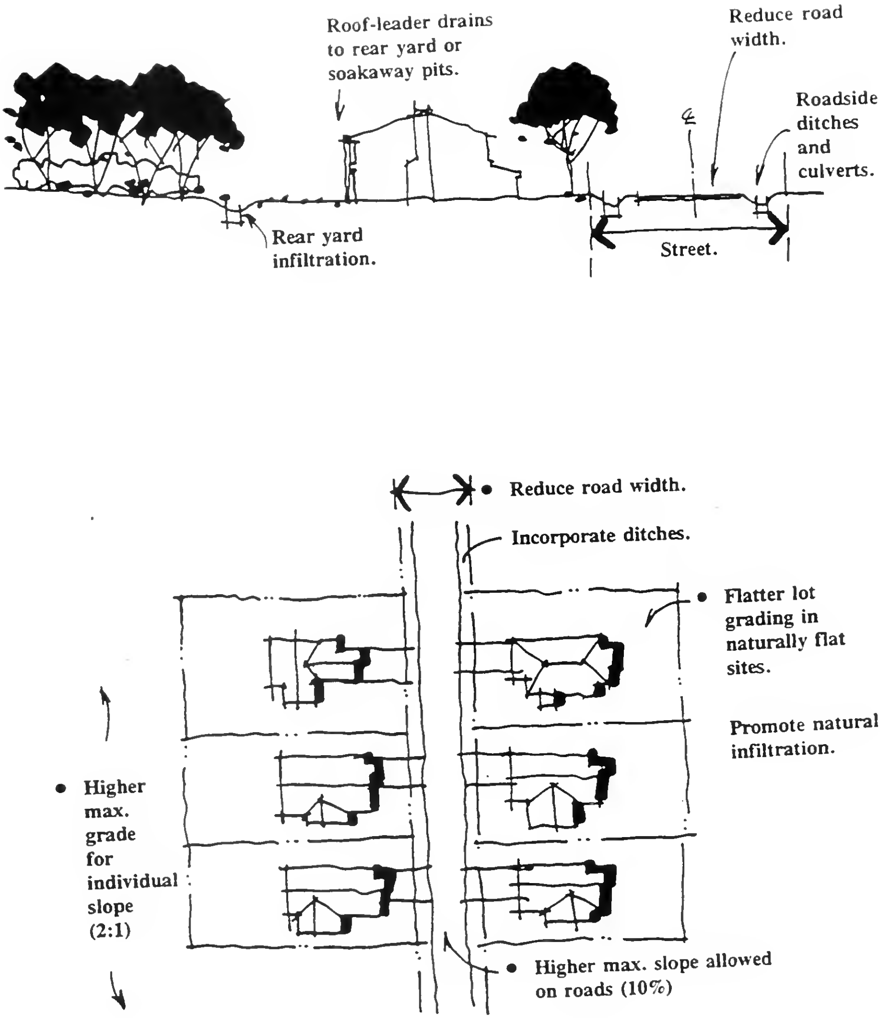


Figure 2:8 Alternative Development Standards

- higher maximum allowable slopes on roads (10% instead of 6%) and individual lots (2:1 instead of 3:1)

  The increase in range of maximum allowable slopes allows planners/engineers greater flexibility in designing developments within the existing topography. Economic and environmental benefits accrue from reduced grading requirements, although there may be some drawbacks such as greater requirements for sanding/salting these roads during the winter and increased erosion potential in roadside ditches. On the other hand, narrower road surfaces will also mean reduced amounts of road salt/sand and lower construction costs. These issues are best addressed from a holistic perspective recognizing the environment, the economy, and the functionality of the subdivision/site design.

- discharge of roof leaders to soakaway pits or rear yards for natural infiltration/evaporation

  Water that is discharged from roof leaders is relatively clean water. The only potential contamination of this water is by atmospheric deposition and roofing materials. Options that promote the infiltration of this water into the surrounding native soil material are promoted since they reduce peak flows and enhance groundwater/baseflow recharge. In areas where infiltration is not appropriate, roof leaders should discharge to the surface as a minimum standard practice.

- servicing via enhanced grassed swales and culverts instead of storm sewers

  The use of grassed swales (commonly referred to as ditch and culvert servicing) is viable for lots which will accommodate swale lengths ≥ the culvert length underneath the driveway (not just the driveway pavement width). The swale length should also be ≥ 5 m for aesthetic and maintenance purposes. This is generally achievable for small lots (9 m) with single driveways or larger lots (15 m) with double driveways. Grassed swales provide numerous benefits (water quality enhancement, reduction of water quantity peak flows and volumes, easier snow removal, storage for snow removal) and are recommended for implementation wherever feasible.

- foundation drains to soakaway pits or sump pumped to the rear yards for natural infiltration

  Foundation drainage is relatively clean water having been filtered by the backfill surrounding the foundation. Options that promote the infiltration of this water into the surrounding native soil material reduce peak flows and enhance groundwater/baseflow recharge. In areas where infiltration is not appropriate, (ie. percolation rate < 15 mm/h) a separate foundation drain should be

considered to reduce the volume of water being treated by any end-of-pipe stormwater management facility. Separate foundation drain collectors should be perforated unless located in areas with a high water table (within 1 metre of the foundation drain).

- increase rear lot overland drainage

  A greater tolerance for designs that allow overland drainage across lots is preferred from an environmental standpoint since they provide greater opportunities for reducing peak flows and stormwater volumes. Overland drainage also provides opportunities for water quality improvement through settling, adsorption, filtration, and infiltration. Opportunities to increase rear lot overland drainage include :

  - allowing lots backing on to one another to drain through each other
  - increasing the allowable length of rear yard swales and contributing drainage area

- increase the allowable vertical sag at intersections (K of 4 instead of 10)

  An increase in the allowable elevation differences for intersection approaches will allow a development to be designed with less changes to the existing topography. This alternative standard is promoted for stop intersections, but may not be applicable for through-type intersections due to increased traffic safety concerns.

Re-create Natural Habitats within the Development Areas

Development activities should be seen as opportunities to re-create lost natural habitats. Within the designated development areas, and as parts of the overall subdivision/site planning concept, re-created ecological habitats should be incorporated within the site. These habitats could be selected areas within public parks, roadside plantings with native woodland species, naturalization of any disturbed slopes, and assisted natural regeneration along existing or new watercourses.

2.4.6 Finalization of the Subdivision/Site Layout

Once the subdivision/site planning and design criteria have been adopted during the preparation of the subdivision/site plan, different subdivision/site layout options may have been generated. These options can include ranges of different design features which comply with the criteria. In order to select a preferred layout, the planners/designers should evaluate the options against the objectives outlined in Section 2.3 above. The subdivision / site layout which best satisfies these objectives should be endorsed as the appropriate development strategy.

2.5 Alterations to a Watercourse

One of the goals of subdivision/site planning is to minimize the alterations to the natural environment as a result of development. It must be recognized that alterations are impossible to avoid in urban areas (road crossings requiring alterations to a watercourse - culvert, bridge, filling, etc.).

In cases where watercourse alterations must occur, the impacts to the environment can be reduced through natural design techniques. Guidance on natural channel design techniques is provided in "Natural Channel Systems : An Approach to Management and Design, Development Draft" (Ministry of Natural Resources, 1994). Readers are directed to this document for more detailed information concerning natural channel designs.

3.0 STORMWATER MANAGEMENT PLAN

3.1 General

The stormwater management plan involves the implementation of stormwater management measures evaluated at the subwatershed plan level. In order to evaluate urban stormwater management practices with respect to receiving water objectives, the level of control, types of SWMPs, and locations of SWMPs should be considered at the subwatershed plan. Ideally, the subwatershed plan will dictate to the stormwater management plan the required types of stormwater management practices and performance level.

The subwatershed plan will not be sufficiently detailed to indicate the location of SWMPs at a plan of subdivision level unless centralized facilities (facilities which serve several plans of subdivision) are proposed. As such, one of the main tasks in a stormwater management plan is the siting of stormwater management practices (Chapter 2).

The siting of SWMPs depends on the types which are proposed. The recommended strategy for stormwater management is to provide an integrated approach to water management that is premised on controlling pollution at the source. **Therefore, a hierarchy of preferred stormwater management practices exists :**

1. Stormwater lot level (source) controls
2. Stormwater conveyance controls
3. End-of-pipe stormwater management facilities

The remaining sections in Chapter 3 provide design guidance on these stormwater management practices. Sections 3.2 through 3.4 provide individual design guidance for lot level, conveyance, and end-of-pipe SWMPs while Section 3.6 illustrates how the individual SWMPs can be assessed at a stormwater management plan level. Section 3.6 provides methods for integrating the SWMP design to achieve water quality, quantity, erosion, and recharge requirements.

3.2 Stormwater Lot Level Controls

Stormwater lot level controls involve measures to treat stormwater before it reaches the subdivision/development conveyance system (storm sewer, swales). Many source controls have been traditionally used for stormwater quantity management such as :

- restricting the number of roof drains to provide rooftop detention of stormwater
- implementing catch-basin restrictors or orifices in the storm sewer to detain stormwater on parking lots
- oversizing storm sewers and implementing orifices in the sewer to create pipe storage

- implementing catch-basin restrictors in rear yard catch-basins to create rear yard storage

All of these measures were designed to detain stormwater to reduce peak runoff rates. They were not designed to reduce the volume of stormwater runoff or treat the quality of stormwater. Generally, the detention timeframe for stormwater quantity measures is in the order of several hours which is too short to significantly enhance water quality. In addition, peak flow control does not address urban impacts on erosion potential and baseflow maintenance.

In order to provide an integrated approach to stormwater management, stormwater lot level controls which preserve the natural hydrologic regime should be implemented. To a certain extent this will also improve the quality of water being discharged to receiving streams. These types of lot level controls include :

- reduced grading to allow greater ponding of stormwater and natural infiltration
- directing roof leaders to rear yard ponding areas or soakaway pits
- sump pumping foundation drains to rear yard ponding areas

The successful implementation of these measures requires innovative subdivision design.

3.2.1 Reduced Lot Grading

Typical development standards require minimum lot grades of 2% for adequate drainage of stormwater away from a building. Alternative Development Standards have been proposed (Ministry of Housing, Ministry of Municipal Affairs, 1994) which reduce the minimum lot grades from 2% to 0.5%. A reduction in the lot grading should be evaluated if the land is naturally flat. In hilly areas, alterations to the natural topography should be minimized (as indicated in Chapter 2).

In order to ensure that foundation drainage problems do not occur, the grading within 2 - 4 metres of a building should be maintained at 2% or higher (local municipal standards should be reviewed to ensure that the grading around a building is in compliance). Areas outside of this boundary should be graded at less than 2 % to create greater depression storage, and promote natural infiltration.

These standards are mainly intended to promote recharge, and reduce the flooding and erosion potential as opposed to enhancing stormwater quality since no road runoff is treated. Increased natural infiltration will help to maintain baseflows, although increased evapo-transpiration may limit the volume of water which is actually recharged.

Design Guidance

Roof Leaders

In areas where flatter lot grading is implemented, roof leaders which discharge to the surface should extend 2 metres away from the building.

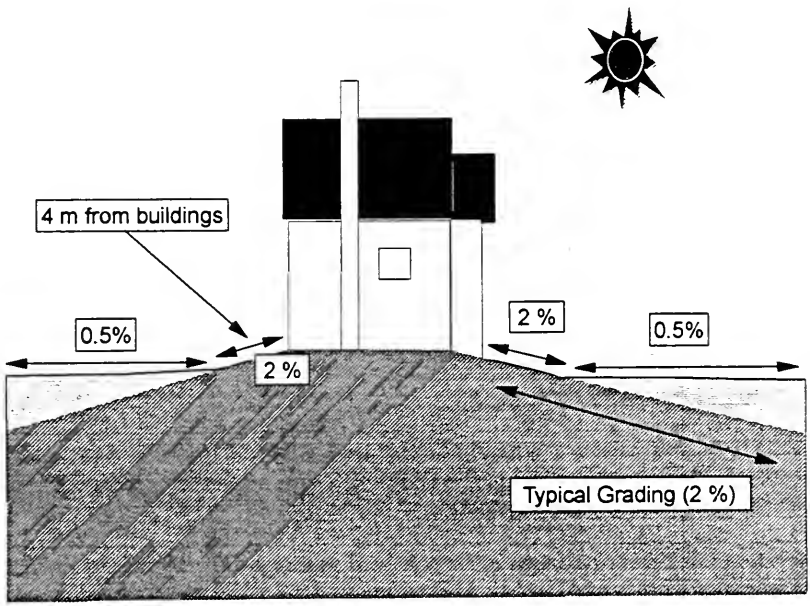


Figure 3.1 Lot Grading Changes

Soil Compaction

Infiltration can be improved by tilling lots with flatter grading to a depth of approximately 300 mm before sod is laid. This would also be of general benefit in all residential areas to address the problems associated with soil compaction (loss of recharge potential) which occurs during construction.

End-of-Pipe Benefits

End-of pipe extended detention requirements for SWMPs will be reduced by decreasing lot grades. The benefits of reduced grades can be assessed based on an increase in pervious area depression storage. It is recommended that the pervious depression storage (or initial abstraction) be increased by 1.5 mm for a change in lot grades from 2% to 0.5% (based on a typical lot 12 × 30 m). Further guidance regarding the reduction in end-of-pipe SWM facility storage for other water management objectives is provided in Section 3.6.

Technical Effectiveness

There is little experience with reduced lot grading as a standard practice on a subdivision scale. The largest impact this practice will have is on the homeowner's utility of his or her land. The water ponded on lots may take 24 to 48 hours to drain which may restrict the active use of the land. However, this impact will be greatest during the spring period, with negligible impacts during the summer period.

It is anticipated that the public will be receptive to this alternative standard if they understand the benefits to be derived from the change in grading practices. As such, this stormwater lot level control is recommended.

3.2.2 Roof Leader to Ponding Areas

The discharge of roof leaders to ponding areas is an additional measure to reduce the potential for downstream flooding and erosion. Water quality benefits from this practice will be limited to the treatment of atmospheric pollutants.

An area for ponding can be created in the rear yard or rear lot line of a lot. Roof leaders are discharged to the surface and directed to the ponding area. Water is detained in the ponding area until it either evaporates or infiltrates.

Design Guidance

Ponding Depth

The area for ponding should be a shallow depression with a maximum depth of 100 mm. An overland flow path should be established for depths greater than this amount.

Roof Leader

The roof leader should discharge into the ponding area via a splash pad and overland flow route.

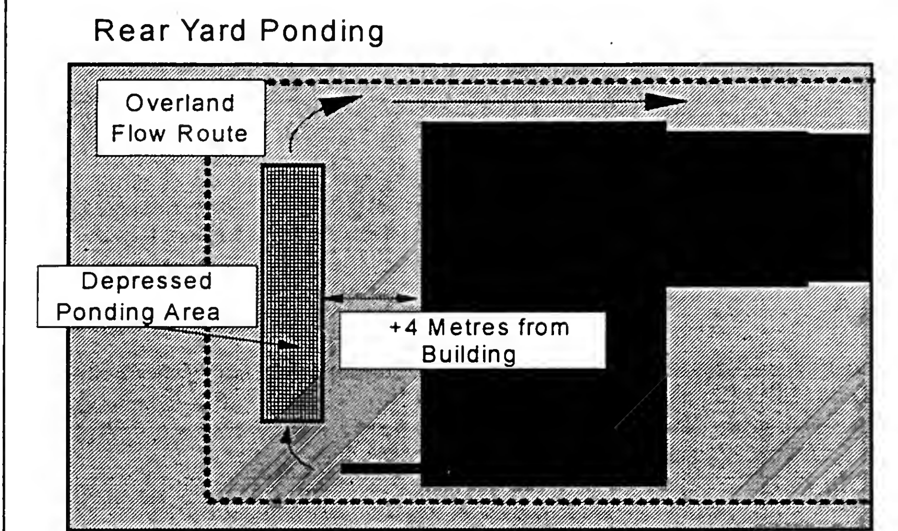


Figure 3.2 Roof Leader Discharge to Rear Yard Areas

Building Setback

The area of ponding should be greater than 4 metres away from any building foundations to ensure that the ponded water does not increase the amount of foundation drainage.

Common Ponding Areas

Ponding areas can be created in the rear lot lines by raising rear yard catch-basins such that they are used as an overflow system. Infiltration in the ponding areas can be enhanced by providing an infiltration trench system underneath the swale (see Soakaway Pits - Section 3.2.3, and Infiltration Trenches - Section 3.4.6).

Compaction

Infiltration can also be improved by tilling the ponding area to a depth of approximately 300 mm before sod is laid. This would also be of general benefit in areas in which ponding is not proposed to address the problems associated with soil compaction (loss of recharge potential) which occurs during construction.

Storage Volume

A minimum storage volume of 5 mm over the rooftop area should be accommodated in the rear yard without overflowing. The maximum target storage volume should be 20 mm over the rooftop area since 90% of all daily rainfall depths are less than this amount (Appendix C).

Soils

A soils report is required to assess the viability of this stormwater lot level control. Lot level ponding can be implemented for soil types with a minimum percolation rate ≥ 15 mm/h. This generally includes all soils coarser than a loam.

Configuration

The configuration of the ponding area will depend on the site specific layout of the development. The depth of ponding should be minimized as a goal. If possible the length of ponding should be maximized compared to the width to prevent short circuiting, to reduce the potential for groundwater mounding, and to maximize the potential for infiltration.

Proximity to Septic Fields

In general surface ponding areas should not be located over Class 4 and 6 sewage system leaching beds to minimize the potential for compaction of the leaching bed and groundwater mounding problems. A hydrogeologist should be consulted with respect to the necessity for mounding calculations and the requirements for a setback from the tile field in areas where surface ponding is proposed. In areas with proposed EPA (Environmental Protection Act) Part VIII Program sewage systems, it will be necessary to consult with the approving Director of the Part VIII program.

End-of-Pipe Benefits

An increase in impervious depression storage is the simplest way to assess the benefits of rear yard storage on the end-of-pipe extended detention requirements. If modelling is performed this would require the rooftop area to be modelled separately from the other areas. The increased depression storage would be equal to the equivalent depth of storage provided over the entire rooftop area. More guidance is provided on the reduction in end-of-pipe SWM facility storage requirements for various water management objectives in Section 3.6.

Commercial and Industrial Roof Storage

In commercial or industrial areas where the buildings have flat roofs, the rooftop hoppers can be raised such that a certain depth of water must be reached before water flows into the hoppers and to the ground below. This effectively provides surface ponding on the roof area itself. The water which is trapped below the hopper opening will evaporate during the inter-event time.

On commercial or industrial roofs, a maximum ponding depth of 10 mm should be allowed before water can flow into the roof hoppers. Calculations must be made during the major system storms to ensure that there are sufficient roof hoppers to limit the maximum ponding depth on the roof to ≤ 65 mm during the 100 year storm.

Technical Effectiveness

This technique has similar benefits and drawbacks to the flatter lot grading. The benefits of this practice, however, outweigh the drawbacks of its implementation, and therefore this stormwater lot level control is recommended.

3.2.3 Roof Leader Discharge to Soakaway Pits

This stormwater lot level control infiltrates roof drainage via an underground infiltration trench. The roof drainage is conveyed directly to the trench by the roof leader. This SWMP provides identical water management benefits to the rear yard ponding option (ie. flooding and erosion potential benefits, and treatment of atmospheric pollutants).

Design Guidance

Water Table Depth

The depth from the bottom of the soakaway pit to the estimated seasonally high water table should be greater than or equal to 1 metre.

Depth to Bedrock

The depth from the bottom of the soakaway pit to bedrock should be greater than or equal to 1 metre.

Building Setback

A typical soakaway pit design is shown in Figure 3.3 The roof leader is extended underground to an excavated infiltration trench. The trench should be located at least 4 metres away from the foundation of the nearest building to prevent excessive foundation drainage.

Storage Media

The trench is comprised of clear stone (50 mm diameter). Non-woven filter cloth should be used to line the trench to prevent the pore space between the stones from being blocked by the surrounding native material.

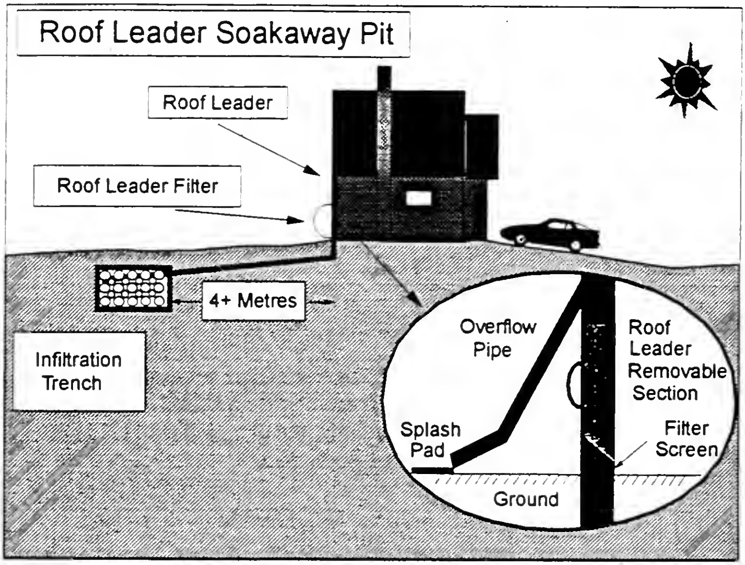


Figure 3.3 Roof Leader Discharge to Soakaway Pit

Conveyance Pipe

The roof leader should extend into the soakaway pit for the full length of the pit. The extension of the roof leader into the pit should be perforated to allow water to fill the pit along the length of the pipe. The perforated pipe should be located near the surface of the trench (75 mm - 150 mm from the top of the pit). A typical trench detail is shown in Figure 3.4.

Soil Cover

Typically the pit should be located close to the ground surface, however this will depend on the depth of storage in the trench, the potential for frost heave, and the stratification of the surrounding soil media. The potential for frost heave is dependent on the surrounding native soils and the potential volume of water in the trench which can freeze. Figure 3.5 provides guidance on the recommended minimum soil cover for various subsurface trench depths and native soil media. This curve has been produced based on professional opinion, the expansion of water due to freezing, and the potential availability of water to freeze. Ice lens formation is not anticipated to occur within the trench due to the size of pores in the storage media. Frost heave and the development of Figure 3.5 are discussed in detail in Appendix D.

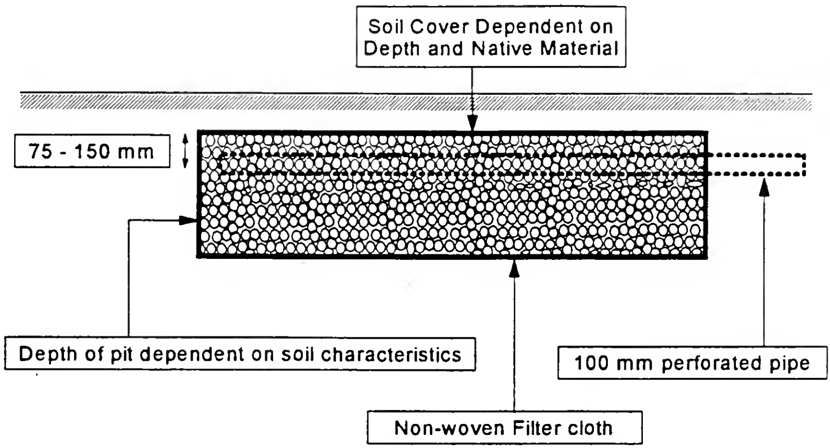


Figure 3.4 Soakaway Pit Details

Soakaway Pit Storage Configuration

The bottom of the pit should extend into the native soil in areas with considerable existing fill (ie. areas where the composition of fill material has not been specified). Shallow soakaway pits are generally preferred since the surficial soils are usually coarser (higher percolation rate) than deeper soils.

The exact depth of the pit, however, will depend to a great extent on the soil stratigraphy of the site. If, for example, a sand lens is located at a depth of 2 metres, it would be advantageous to construct a deep soakaway pit which drains into the lens.

The length and width of a soakaway pit are dependent on the configuration of the development. The length of trench (in the direction of inflow) should be maximized compared to the width to ensure the proper distribution of water into the entire trench and to minimize the potential for groundwater mounding (Groundwater mounding is a local increase in the water table due to the infiltration of water and is more prevalent if a greater volume of water infiltrates in a localized area - ie. square trenches will have greater groundwater mounding).

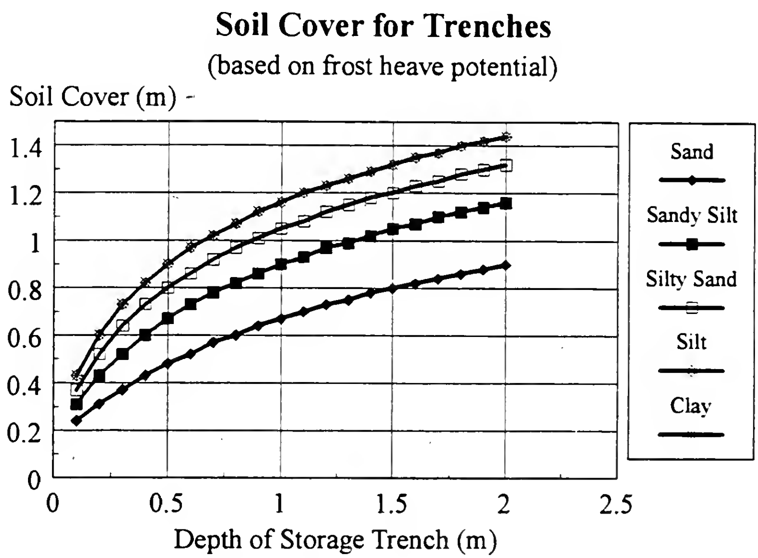


Figure 3.5 Soil Cover for Underground Trenches

The permeability of the native soil will dictate the maximum allowable underground storage depth as indicated by Equation 3.1. Storage depths greater than 1.5 m are generally not recommended for soakaway pits from both a cost, and a compaction perspective. The weight of the water in a deep soakaway pit will compact the surrounding native soil and decrease the infiltration capacity.

There are exceptions, however, to this maximum depth recommendation. In areas with deep sand lenses or significant horizontal soil stratification, deep soakaway pits may be preferred. Soils investigations should be undertaken to determine whether these situations exist.

$$d = P\,T$$ Equation 3.1 Maximum Allowable Soakaway pit depth

where d = maximum allowable depth of the soakaway pit (m)
P = percolation rate (Table 3.1) (m/h)
T = drawdown time (24 - 48 h) (h)

It is recommended that a conservative drawdown time (24 h) be chosen recognizing that the

percolation rates into the surrounding soil will decrease over time and that there will likely be a lack of maintenance since it is located on private property.

Table 3.1 Minimum Soil Percolation Rates	
Soil Type	Percolation Rate (mm/h)
sand	210
loamy sand	60
sandy loam	25
loam	15

Storage Volume

A minimum storage volume of 5 mm over the rooftop area should be accommodated in the soakaway pit without overflowing. The maximum target storage volume should be 20 mm over the rooftop area since 90% of all daily rainfall depths are less than this amount (Appendix C).

Common Soakaway Pits

Common soakaway pits may be viable in areas with compact build forms. The common soakaway pit can be located in neighbourhood park area or along rear lot lines.

Proximity to Septic Fields

Groundwater mounding calculations may be required to ensure that soakaway pits do not interfere with Class 4 and 6 sewage system leaching beds. A hydrogeologist should be consulted with respect to the necessity for mounding calculations and the requirements for a setback from the tile field. It is anticipated that calculations will be required in areas where the soils are marginally acceptable for infiltration. In areas with proposed EPA (Environmental Protection Act) Part VIII Program sewage systems, it will be necessary to consult with the approving Director of the Part VIII program. In areas where the percolation rate is high (≥ 50 mm/h) groundwater mounding, and hence interference, will generally not be a problem.

Soils

A soils report is required to assess the viability of this type of stormwater lot level control. Soakaway pits can be implemented for soil types with a minimum percolation rate ≥ 15 mm/h. This generally includes all soils coarser than a loam.

Overflow By-pass

An overflow pipe should be installed from the roof leader to discharge to a splash pad. A removable filter should be incorporated into the roof leader below the overflow pipe. The filter should have a screened bottom to prevent leaves and debris from entering the soakaway pit. It should be easy to remove so that a homeowner can clean the filter. Frequent use of the overflow pipe will indicate the need for filter screen maintenance.

Technical Effectiveness

Soakaway pits for roof leader drainage have been implemented in numerous areas (eg. Toronto, Maryland). In a recent study (Lindsey et al., 1992) monitoring indicated that 60% of 25 soakaway pits which were studied were operating as designed.

Soakaway pits have both benefits and drawbacks compared to rear yard ponding. The benefits include greater recharge (less evapotranspiration) and less inconvenience to the homeowner (less surface water ponding). The drawbacks include greater maintenance and uncertain longevity.

The potential for clogging (maintenance problems) is reduced compared to end-of-pipe infiltration techniques (infiltration basins, trenches, pervious pipes) since soakaway pits only accept roof drainage (roof drainage contains less suspended solids than road runoff). Accordingly, this SWMP is recommended for general implementation.

3.2.4 Sump Pumping of Foundation Drains

Current development standards allow foundation drains to be connected to the storm sewer. Proposed alternative standards (Ministry of Housing, Ministry of Municipal Affairs, 1994) allow the use of sump pumps to discharge foundation drainage to either the surface or soakaway pits. Either of these two options is preferable to the connection of foundation drains to either the storm or sanitary sewer.

Foundation drainage is relatively clean water, and as such, should be kept separate from polluted water (storm drainage and sanitary drainage). This will reduce both the cost of stormwater management (quality, erosion, quantity) and sewage treatment.

Design Guidance

Water Table Depth

Although preferable, foundation drainage by sump pumps is not always feasible.

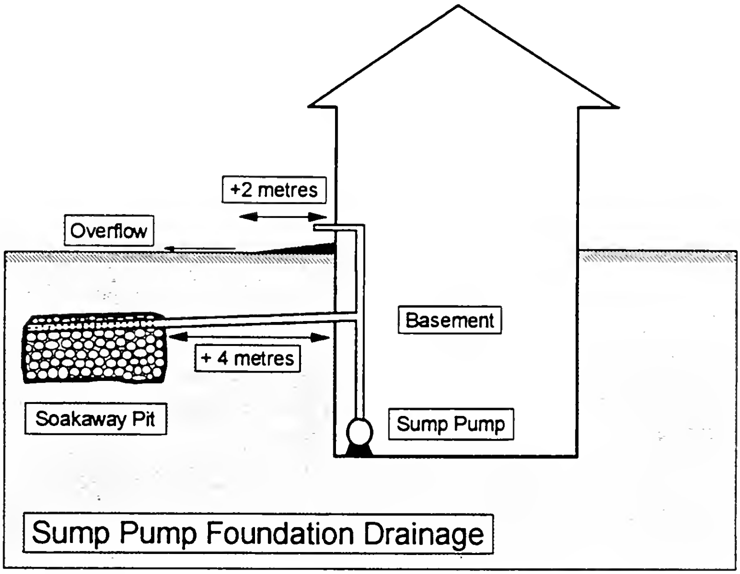


Figure 3.6 Foundation Drainage Options

In areas where the seasonally high water table is within 1 metre of the building foundation drains, sump pumps should not be utilized. This requirement is imposed to prevent excessive sump pump operation in areas with high water tables and to prevent a looped system whereby the sump pump discharges maintain the foundation drainage. In these areas, a separate (third pipe) pipe should convey foundation drainage to the receiving water.

<u>Depth to Bedrock</u>

In areas where the depth to bedrock is within 1 m of the foundation drain elevation, foundation drainage by sump pumps is not feasible. This requirement is imposed to prevent excessive sump pump operation and a looped system.

<u>Building Setback</u>

Figure 3.6 demonstrates foundation drainage to a soakaway pit using a sump pump. If a soakaway pit is used it should be located a minimum of 4 metres away from all building foundations to minimize the contribution of soakaway pit drainage to foundation drainage. If the foundation drains are being discharged directly to the surface, the discharge point at the ground surface should be located a minimum of 2 metres away from all building foundations.

In the case of a discharge to the surface, it should be ensured that there is sufficient grade from the foundation wall away from the building (≥ 2%) for 2 - 4 metres to convey the foundation drainage away from the building.

Overland Flow

Discharges to the surface should be directed to the rear yard to minimize the amount of surface drainage over sidewalks during the winter. Sump pumps discharging to the surface should discharge approximately 0.5 m above the ground surface to prevent blockages in the winter due to ice and snow.

3.3 Stormwater Conveyance Controls

Stormwater conveyance controls are implemented as part of the stormwater conveyance system. Stormwater is conveyed from developed areas by either sewers or grassed swales. Stormwater conveyance controls can be classified into three categories :

- pervious pipe systems
- pervious catch-basins
- grassed swales

3.3.1 Pervious pipe systems

A few municipalities (eg. City of Nepean, City of Etobicoke) in Ontario have implemented pervious pipe systems for stormwater drainage. It should be noted that these systems are still experimental in nature and have experienced some problems in the past.

Pervious pipe systems are perforated along their length allowing exfiltration of water through the pipe wall as it is conveyed downstream. The pipe itself is similar to that used for tile drainage on agricultural lands and is available with either a smooth-walled or corrugated interior.

Design Guidance

There are three main requirements for pervious pipe systems :

- pre-treatment of road runoff
- soils with good infiltration potential
- deep groundwater table

Pre-treatment

Pervious pipe systems are intended to convey road drainage which has high levels of suspended sediment. It is important that some form of pre-treatment be implemented for road drainage before it reaches the pervious pipe system.

Exfiltration of stormwater without pre-treatment will result in poor longevity of the pervious pipe system, and may require a groundwater impact assessment under the MOEE Reasonable Use Policy.

Pre-treatment of pervious pipe systems can best be achieved by the incorporation of grassed boulevards as pre-treatment areas (Figure 3.7). Stormwater is conveyed from the road to a low boulevard. The boulevard is graded towards catch-basins which are connected to the pervious pipe system. The catch-basins are raised such that water must reach a certain depth in the boulevard before it can overflow into the pervious pipe system. This will provide a sense of an urban cross-section while maintaining the benefits of traditional grass surfaced conveyance systems.

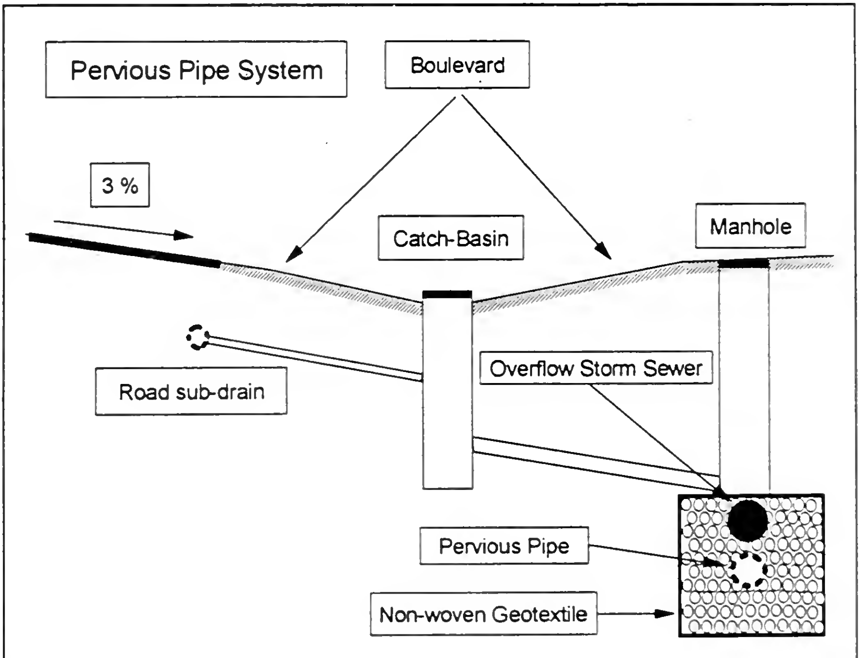


Figure 3.7 Pervious Pipe System

Soils

Soils information is required for the design of pervious pipe systems. Pervious pipes should be implemented in soils with a percolation rate ≥ 15 mm/h.

Water Table Depth

If a pervious pipe system is implemented in an area with a seasonally high water table which is higher than the obvert of the pipe, the pipe will drain the groundwater table. In this scenario, depending on the native soil characteristics and whether the trench or pipe is wrapped in geotextile fabric, soil can be transported into the pipe system undermining the pipe foundation and leading to structural failure. In order to ensure this does not happen pervious pipe systems should not be implemented in areas where the seasonal high groundwater level is within 1 metre of the bottom of the storm sewer backfill.

Depth to Bedrock

The depth to bedrock should be greater than or equal to 1 metre below the bottom of the perforated pipe storage media to ensure adequate drainage/hydraulic potential.

Pipe Slope

Pervious pipe systems should be implemented with reasonably flat slopes (0.5 %) to promote exfiltration.

Geotextiles

Although a filter sock can be used to prevent fines from entering the pipe system from the native material, the sock may prevent fines in stormwater from exfiltrating to the native material. As such, the use of a filter sock may cause clogging at the pipe/sock interface and decrease the longevity of the pervious pipe system. Therefore, its implementation should be discussed with the local municipality.

Non-woven filter fabric should be implemented at the interface between the pipe bedding (exfiltration storage) and the native soil to prevent native material from clogging the voids in the exfiltration storage media.

Pervious Pipe Bedding/Storage Media

Granular A material, or preferably clear stone (50 mm) should be used for the pipe bedding. Granular B bedding has too many fines which may infiltrate the pipe system and is generally discouraged for use as pervious pipe bedding.

Pervious pipe systems should be constructed with anti-seepage collars along the bedding to

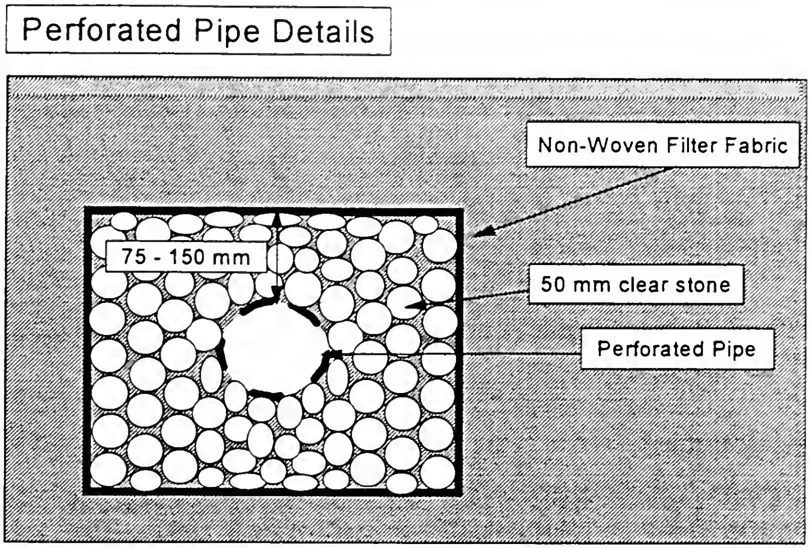


Figure 3.8 Pervious Pipe Detail

ensure that exfiltrated water does not travel along the pervious pipe bedding to the pipe outlet. The spacing of anti-seepage collars is left to the professional opinion of the engineer but should be based on the permeability of the native soil material and the pipe slope.

Storage Volume

A minimum storage volume equal to the runoff from a 4 hour 5 mm storm over the contributing drainage area should be accommodated in the pervious pipe bedding/storage media without overflowing. The maximum target storage volume should be equal to the runoff from a 4 hour 15 mm storm over the contributing drainage area since 80% of all daily rainfall depths are less than this amount (Appendix C).

Storage Configuration

The exfiltration storage bedding layer should be 75 - 150 mm deep above the pervious pipe. A shallow bedding above the pipe is used since the storage above the pipe obvert is not utilized. The depth of bedding below the pipe obvert is dependent on the storm to be exfiltrated and the surrounding native soil material. Maximum depths which permit the bedding to drain in 24

hours can be calculated using Equation 3.1. A detail of a pervious pipe end section is shown in Figure 3.8.

The width of storage can be determined based on the rate of exfiltration from the pipe, the proposed length of pervious pipe, and the desired volume of runoff for infiltration. Section 3.6 provides methods for estimating the rate of exfiltration based on the number and size of pipe perforations.

Pervious Pipe

Smooth-walled (interior) pervious pipe is recommended for stormwater exfiltration since corrugated pipe has a higher potential for clogging. Furthermore, the maintenance of corrugated pipe via traditional sewer flushing is relatively ineffective since material becomes trapped in the corrugations. A minimum diameter of 200 mm should be used for the pervious pipe to facilitate maintenance.

Technical Effectiveness

Pervious pipe systems have historically been unreliable in terms of long term performance. In areas where they have been implemented and monitored, numerous systems were noted to have clogged after several years (Wisner et al. 1990, Delcan 1988, Marshall Macklin Monaghan Limited, 1991). Clogging can be attributed to many different factors :

- poor design (storage media, lack of filter cloth, lack of pre-treatment)
- poor construction practices
- inadequate stabilization of development before implementation of pervious pipe (construction timing)
- poor site physical conditions (soils, water table)

One of the problems of implementing pervious pipe systems is construction timing. The pervious pipe system functions as the storm sewer and therefore must be constructed in its entirety at a certain point in time. Ideally, for new development it would be constructed after the houses have been built and sod has been laid. However, the road sub-grade needs to be drained and would require the pipe system to be implemented with the road network. Although the catch-basins can be blocked to try and prevent sediment laden water from clogging the pervious pipe system, there is a great potential for clogging and compaction of the system during the construction phase of development.

The exfiltration of surface road drainage is a controversial issue recognizing the elevated levels of metals, oil/grease, and chlorides in this water. The MOEE Reasonable Use Policy may apply, however, its application is usually site specific and dependent on whether there is the potential for exfiltrated stormwater to contaminate a deep groundwater aquifer system, or a shallow system if it is being used for drinking water. If there is reasonable certainty that the

exfiltrated water will only enter a shallow groundwater system which discharges to a nearby surface stream, Reasonable Use is generally not being applied by the MOEE Regional staff.

There is the potential for long term clogging of pervious pipe systems without pre-treatment and maintenance. Pre-treatment is difficult given the compact form of development which is being promoted in the province today. Therefore, although acceptable, this stormwater conveyance control is not encouraged for widespread use in new development. Pervious pipe systems are more applicable in retrofit situations where there are land constraints and the site is stabilized.

One municipality (City of Etobicoke) has implemented a double pipe system (regular storm sewer over a perforated pipe) in a retrofit situation on a local road which is not subject to heavy salting or sanding. This system, although more expensive, provides a contingency conveyance system if the perforated pipe becomes clogged. A double pipe system also allows the perforated pipe to be plugged during the construction phase until the site has stabilized, thereby preventing it from becoming clogged prematurely. This system is fairly new and has been designed with several features to enhance maintenance. The City has implemented a monitoring program to determine its utility and future application in City projects.

3.3.2 Pervious Catch-basins

Pervious catch-basins are simply normal catch-basins with a larger sump which are physically connected to exfiltration storage media. In some designs the storage media is located directly beneath the catch-basin via a hole or series of holes in the catch-basin floor. Although this design is convenient and conserves land, it is more susceptible to clogging and compaction as a result of the lack of pre-treatment and weight of water in the catch-basin. There are manufacturers which offer catch-basin filters as a pre-treatment media in this type of design. These filters are expensive, however, and need frequent replacement.

A second design (Figure 3.9) uses the catch-basin sump for pre-treatment of runoff and discharges low flows through the wall of the catch-basin to the exfiltration storage media located besides the catch-basin.

Design Guidance

Pre-treatment

Pervious catch-basins are intended to infiltrate road drainage which has high levels of suspended sediment. It is important that some form of pre-treatment be implemented for road drainage before it reaches the pervious catch-basin. Exfiltration of stormwater without pre-treatment will result in poor longevity of the exfiltration system, and may require a groundwater impact assessment under the MOEE Reasonable Use Policy.

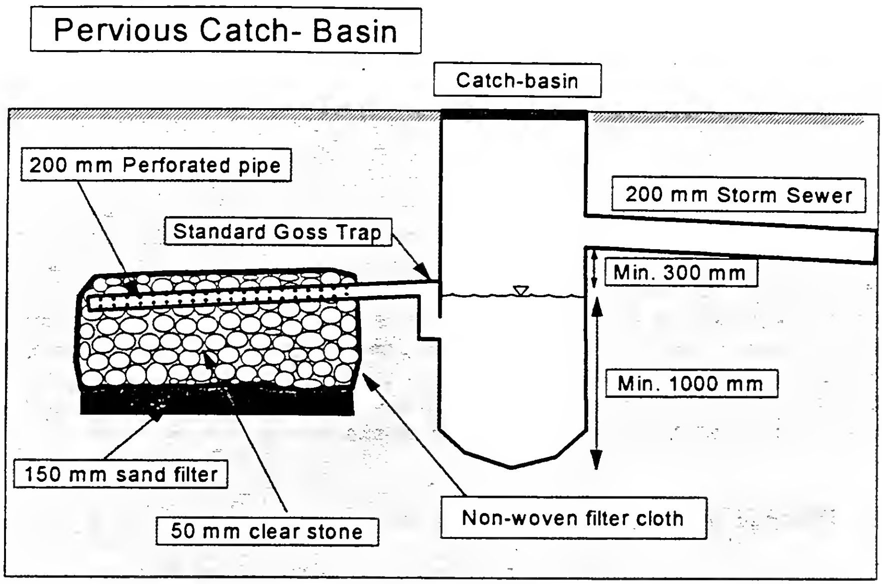


Figure 3.9 Pervious Catch-Basin

Pre-treatment is best achieved by the incorporation of grassed boulevards as pre-treatment areas (Figure 3.7). This right of way configuration was discussed in the previous section on pervious pipes.

Oversized catch-basins will provide pre-treatment for systems similar to that shown in Figure 3.9. Large catch-basins with deep sumps will help to pre-treat the runoff before it is conveyed to the infiltration trench (it should be recognized, however, that even with large manholes the amount of pre-treatment will be small, and that other pre-treatment measures (ie. grassed swales/filter strips) should be incorporated, if possible, before the stormwater enters the sewer system).

Soils

Soils information is required for the design of pervious catch-basins. Pervious catch-basins should be implemented in soils with a percolation rate ≥ 15 mm/h.

Water Table Depth

Pervious catch-basins should not be implemented in areas where the seasonal high groundwater

level is within 1 metre of the bottom of the infiltration trench.

Depth to Bedrock

The depth to bedrock should be greater than or equal to 1 metre below the bottom of the infiltration trench to ensure adequate drainage/hydraulic potential.

Geotextile

Non-woven filter fabric should be implemented at the interface between the exfiltration storage and the native soil to prevent native material from clogging the voids in the exfiltration storage media.

Storage Volume

A minimum storage volume equal to the runoff from a 4 hour 5 mm storm over the contributing drainage area should be accommodated in the pervious pipe bedding/storage media without overflowing. The maximum target storage volume should be equal to the runoff from a 4 hour 15 mm storm over the contributing drainage area since 80% of all daily rainfall depths are less than this amount (Appendix C).

Storage Media

Clear stone (50 mm) should be used as the exfiltration storage media (porosity = 0.4).

Storage Configuration

The exfiltration storage depth is dependent on the native soil type/characteristics. Maximum depths can be calculated based on the native soil percolation rate and Equation 3.1. The length and width will depend on the area of land available for the trench (up to the maximum storage volume equal to the runoff from a 15 mm storm over the contributing area).

Technical Effectiveness

Pervious catch-basins are experimental in nature and have not undergone extensive evaluation. It is expected, however, that the potential for clogging is similar to pervious pipe systems and is related to :

- poor design (storage media, filter cloth, lack of pre-treatment)
- poor construction practices
- inadequate stabilization of development before construction (construction timing)
- poor site physical conditions (soils, water table)

One of the benefits of pervious catch-basins which are located off-line is that they can be plugged until construction has finished and the development has been stabilized. This helps to prolong the life of the exfiltration storage.

As with pervious pipe systems, the exfiltration of surface road drainage is a contentious issue recognizing the elevated levels of metals and oil/grease in this drainage. The MOEE Reasonable Use guidelines may apply to some development applications which propose pervious catch-basin designs. The application of Reasonable Use is usually site specific and dependent on whether there is the potential for exfiltrated stormwater to contaminate a deep groundwater aquifer system, or a shallow system if it is being used for drinking water. If there is reasonable certainty that the exfiltrated water will only enter a shallow groundwater system which discharges to a nearby surface stream, Reasonable Use is generally not being applied by the MOEE Regional staff.

Long term clogging as a result of a lack of pre-treatment and catch-basin maintenance is the major drawback of this stormwater conveyance control. Frequent catch-basin cleaning is required to ensure the longevity of this SWMP. Ultimately, however, the exfiltration storage will become clogged and will need to be replaced. Therefore, although acceptable, this stormwater conveyance control is not encouraged for widespread use at this time.

3.3.3 Grassed Swales

Grassed swales are generally associated with rural drainage. This association was started as numerous urban areas began endorsing the dual drainage concept approximately twenty years ago. Since that time stormwater management objectives have changed and grassed swales are being promoted to filter and detain stormwater runoff.

Design Guidance

Swale Cross-Section

Grassed swales can be effective SWMPs for pollutant removal if designed properly. The water quality benefits associated with grassed swales depend on the contact area between the water and the swale and the swale slope. Deep narrow swales are less effective for pollutant removal compared to shallow wide swales. Given typical urban swale dimensions of a 0.75 metre bottom width, 2.5:1 side slopes, and a 0.5 metre depth, this generally limits the contributing drainage area to $\leq$ 2 ha (to maintain a flow $\leq 0.15\ m^3/s$ and a velocity $\leq$ 0.5 m/s). Table 3.2 indicates drainage area restriction guidelines based on percent imperviousness. The values noted in Table 3.2 are guidelines based on the assumptions given and will change based on the channel cross-section, slope, and cover.

Grassed swales are effective for stormwater treatment as long as a minimum channel slope is

maintained (≤ 1%) and a wide bottom width (≥ 0.75 m) is provided. Grass should be allowed to grow higher than 75 mm to enhance the filtration of suspended solids.

The swales evaluated in Table 3.2 are indicative of swales servicing an urban subdivision and not a transportation corridor. In general, the width of grassed swales should be maximized, and the slope and depth of flow minimized, to maximize the water quality benefits.

Table 3.2 Grassed Swale Drainage Area Guidelines*

% Imperviousness	Maximum Drainage Area (ha)
35	2.0
75	1.5
90	1.0

* based on the following assumptions (trapezoidal channel, grassed lined (n=0.035), slope of drainage area = 2%, 2.5:1 side slopes, 0.75 m bottom width, 0.5% channel slope, max. allowable Q=0.15 m^3/s, max. allowable V= 0.5 m/s)

Flow Velocity

As a general guideline, grassed swales designed for water quality enhancement should be designed to convey the peak flow from a 4 hour 25 mm Chicago storm with a velocity ≤ 0.5 m/s. This guideline results in a requirement for wide, flat swales for larger drainage areas.

All grass swales must be evaluated under major system and minor system events to ensure that the swale can convey these storms effectively.

Ditch and Culvert Servicing

Ditch and culvert servicing is viable for lots which will accommodate swale lengths ≥ the culvert length underneath the driveway (not just the driveway pavement width). The swale length should also be ≥ 5 m for aesthetic and maintenance purposes. This is generally achievable for small lots (9 m) with single driveways or larger lots (15 m) with double driveways.

Maintenance and Performance Enhancements

In order to promote infiltration of stormwater and the settling of pollutants, permanent check dams can be constructed at intervals along the swale system. These enhancements are best utilized on large swales where the cumulative flow depth and rate is not conducive to water quality enhancement ($V \geq 0.5$ m/s & $Q \geq 0.15$ m^3/s during the 25 mm 4 hour storm). The distance between check dams can be calculated based on the depth of water at the check dam and the swale channel slope. For example, a swale with a 1% slope and a check dam height of 0.3 m, the distance between check dams should be 30 metres (or less). Figure 3.10 illustrates an enhanced grass swale design.

The dam should be constructed out of durable material (wood) which blends into the surrounding landscape. A rock check dam can be used if the swale is located in a remote area which is not subject to vandalism. The dam should be configured in a V shape to help minimize scour and erosion of the downstream swale banks (V points upstream). The dam should be securely

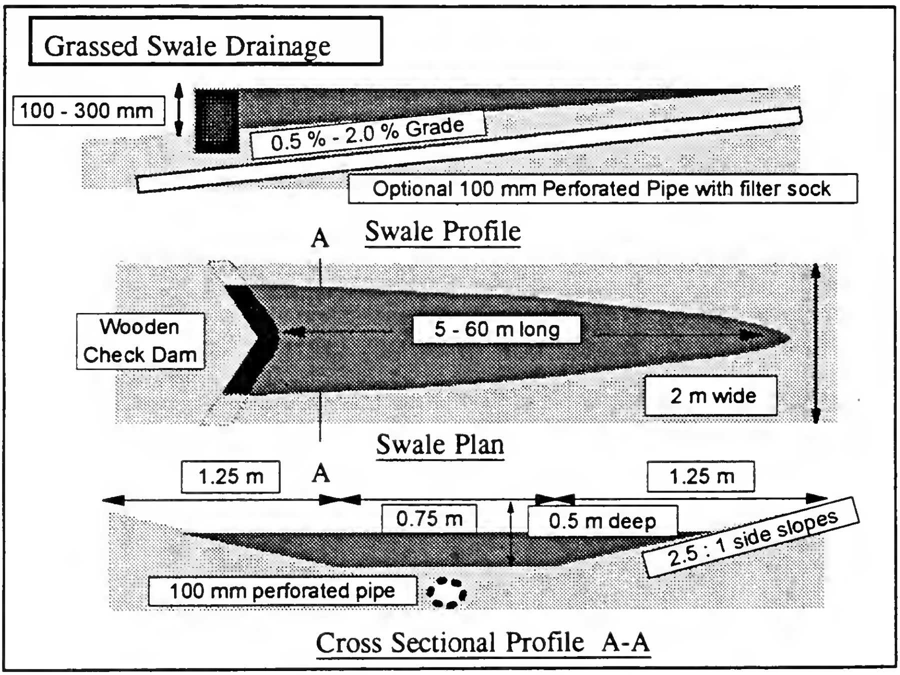


Figure 3.10 Enhanced Grass Swale Design

embedded in the swale banks and some rip rap should be placed downstream of the dam to prevent scour and erosion. Velocities of the design conveyance storm should be kept to approximately 1 m/s whereby smaller stone sizes can be utilized (75 mm diameter).

In areas where the swales are separated by driveway culverts, the culverts can be raised such that the driveway embankment (up to the invert of the driveway culvert) acts as the check dam. This design is more aesthetically appealing and negates the need for rip rap erosion protection. The driveway culvert should be underdrained, however, to ensure that a permanent pool of water is not created in the swale over time.

A low flow opening can be created in the check dam to ensure a drawdown time $\leq$ 24 hours. Recognizing the potential for clogging of the low flow opening, however, it is recommended that swales with check dams be underdrained in soils with poor infiltration potential (eg. clays).

Standard 100 mm perforated pipe (or larger) should be used in combination with a filter sock in any type of underdrain system. Stone storage can be provided around perforated pipes that are implemented under swales as a secondary storage media to promote exfiltration. The appropriate depth of soil cover for the stone storage should be based on the surrounding soil conditions and the potential for frost heave. Figure 3.5 indicates the recommended soil cover based on the surrounding native soil type and trench depth.

All grass swales must be evaluated under major system and minor system events neglecting the storage/conveyance below the overflow of any check dam to ensure that the swale can convey these storms effectively.

3.4 End-of-Pipe Stormwater Management Facilities

End-of-pipe stormwater management facilities receive stormwater from a conveyance system (ditches, sewers) and discharge the treated water to the receiving waters. There are nine generalized categories of end-of-pipe SWM facilities :

- wet ponds
- wetlands
- dry ponds
- infiltration basins
- infiltration trenches
- filter strips
- buffer strips
- sand filters
- oil/grit separators

3.4.1 Terminology

This section describes the terminology that will be used in this manual for end-of-pipe stormwater management facilities. This section is provided since there can be confusion with respect to the terminology used in the field of stormwater management. Most of the confusion revolves around the terminology used to describe wet facilities. For example, numerous terms are used to describe wet ponds :

- wet pond
- extended detention wet pond
- micro pool extended detention wet pond

These ponds are variations of one another, and will be discussed in this manual as one type of SWMP.

Extended Detention Storage

Extended detention storage refers to the dry or active storage provided in a SWMP. The difference between a "wet pond" and an "extended detention wet pond" is that the former discharges water at the same rate as water is conveyed to it, while the latter releases water at a slower rate compared to the influent water. The extended detention storage, or active storage, provides benefits with respect to water quality, erosion potential, and flooding potential.

In Ontario, all wet ponds incorporate extended detention storage (active storage) since it is a requirement of the MOEE/MNR Interim Guidelines for Stormwater Quality (MOEE/MNR, 1990 ,1991). Accordingly, all dry ponds, wet ponds, and wetlands designed in Ontario are extended detention dry ponds, extended detention wet ponds and extended detention wetlands, respectively. Therefore, this manual will henceforth refer to these facilities simply as wet ponds, dry ponds, and wetlands. The extended detention storage component is considered to be implied by the nature of the historical design requirements imposed in the province.

Permanent Pool

The permanent pool is the portion of a wet facility which never drains (except during maintenance). Historically, there have not been any design requirements concerning the permanent pool storage in wet facilities.

The permanent pool provides two functions. During a storm event the permanent pool acts as a buffer such that any water which is being discharged from the facility is either clean or diluted. After the storm, pollutants remain trapped in the permanent pool. These pollutants have on the order of 72 hours to settle before another rainfall event occurs (72 hours is the average inter-event time estimated in Ontario (Gietz, 1983)). This inter-event settling is one of the main reasons why wet ponds are more effective in pollutant removal than dry ponds.

Design guidance is provided in Chapter 4 of this manual with respect to sizing the permanent pool in wet type SWMPs recognizing its important function.

3.4.2 Location of End-of Pipe Stormwater Management Facilities

Urban SWMPs should be located outside of the floodplain (above the 100 year elevation) wherever possible. In some site specific instances, SWMPs may be allowed in the floodplain if there is sufficient technical or economic justification and given that they meet certain requirements :

- The cumulative effects resulting from changes in floodplain storage, and balancing cut and fill, do not adversely impact existing or future development
- Effects on corridor requirements and functional valleyland values must be assessed. SWMPs would not be allowed in the floodplain if detrimental impacts could occur to the valleyland values or corridor processes.
- The SWMPs must not affect the fluvial processes in the floodplain.
- The outlet invert elevation of the SWMP should be higher than the 2 year floodline and the overflow elevation must be above the 25 year floodline.
- An online stormwater quantity facility would be acceptable if designed such that the bank full flows, and hence fish movement, are not impeded/obstructed, and that the foregoing requirements noted above are met. An online facility could only be proposed in the context of a subwatershed plan.

The location of end-of-pipe stormwater management facilities is a contentious issue since the use of tableland reduces the overall developable area. In an effort to minimize the loss of developable land municipalities should allow the use of park land dedication for SWMPs which offer passive recreational opportunities and follow the municipality's greenland strategies (parkland objectives) wherever possible.

3.4.3 Wet Ponds

Extended detention wet ponds are the most reliable end-of-pipe stormwater management facility to-date in terms of pollutant removal. This reliability can be attributed to several factors :

- performance does not depend on soil characteristics
- the permanent pool prevents re-suspension
- the permanent pool minimizes blockage of the outlet
- biological removal of pollutants
- the permanent pool provides extended settling

Unfortunately, wet ponds also have environmental impacts related to increased downstream water temperature which may limit their application in certain areas.

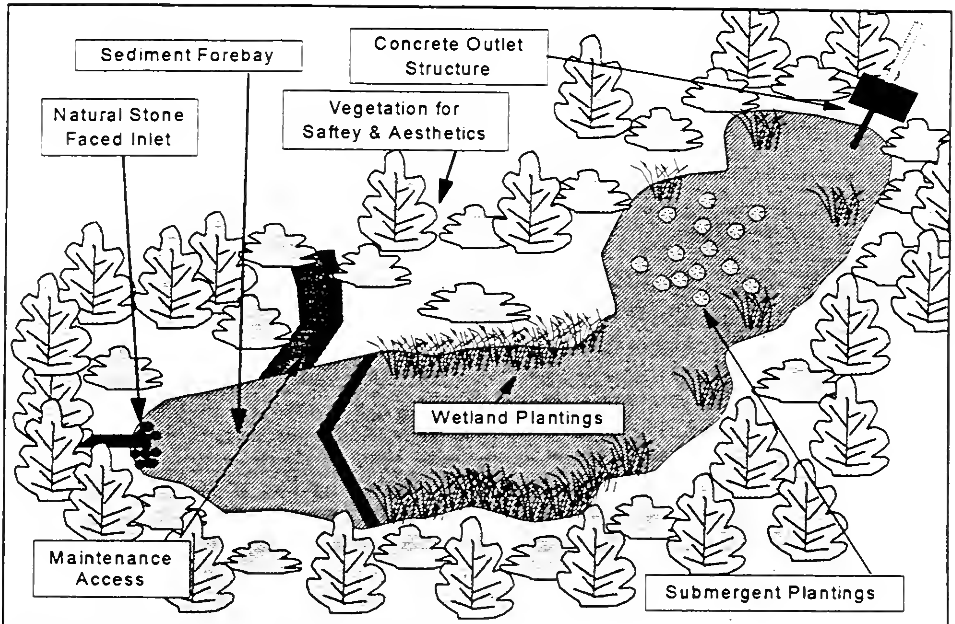


Figure 3.11 Extended Detention Wet Pond

Design Guidance

Common sense guidelines have the greatest impact on the performance of a wet pond (or any other SWMP for that matter). These guidelines can be classified into several groups :

- drainage area
- length to width ratio
- permanent pool and active storage depth
- planting strategy
- inlet and outlet configurations
- maintenance and performance enhancements

Sizing

In areas with subwatershed planning, the plan will provide guidance with respect to the active storage (extended detention) sizing. Guidance will also be provided indirectly regarding the permanent pool since the subwatershed plan will indicate the uses/concerns associated with the receiving waters (should identify habitat classification). If a subwatershed plan has not been,

and will not be, undertaken, the sizing of both storage components is detailed in Chapter 4. Sizing of these components was based on long term continuous modelling of suspended solids removal performance (Appendix I).

Drainage Area

Wet ponds require a minimum drainage area to sustain the permanent pool. As a general rule wet ponds should be implemented for drainage areas $\geq$ 5 hectares.

Detention Time

A detention time of 24 hours should be targeted in all instances, unless the potential for clogging the outlet is high.

In cases where the outlet is susceptible to clogging (ie. drainage areas < 8 ha - see minimum orifice size) the detention time can be reduced to a minimum of 12 hours. (Note : simulations that were performed in this study indicated that a 12 hour detention time produced similar settling to a 24 hour detention time)

The drawdown time in the pond can be estimated using Equation 3.2. Equation 3.2 is the classic falling head orifice equation which assumes a constant pond surface area. This assumption is generally not valid and a more accurate estimation can be made if Equation 3.2 is solved as a differential equation. This is easily done if the relationship between pond surface area and pond depth is approximated using a linear regression. A discussion of this equation is provided in Appendix E, and an example of the calculation of drawdown time is provided in Chapter 8.

$$t = \frac{2\ A_p}{CA_o(2g)^{0.5}} \quad (h_1^{0.5} - h_2^{0.5})$$

Equation 3.2 Drawdown time

or if a relationship between A_p and h is known (ie. $A = C_2h + C_3$)

$$t = \frac{0.66C_2\ h^{1.5} + 2C_3h^{0.5}}{2.75\ A_o}$$

where
- t = drawdown time in seconds
- A_p = surface area of the pond
- C = discharge coefficient (typically 0.63)
- A_o = cross-sectional area of the orifice (m^2)
- g = gravitational acceleration constant (9.81 m/s^2)
- h_1 = starting water elevation above the orifice (m)
- h_2 = ending water elevation above the orifice (m)
- h = maximum water elevation above the orifice (m)
- C_2 = slope coefficient from the area-depth linear regression

C_3 = intercept from the area-depth linear regression

Minimum Orifice Size

The smallest diameter orifice accepted by most municipalities to ensure that clogging does not occur in a stormwater system is 75 mm. It is recommended that this minimum size be maintained for exposed outlet designs (ie. reverse sloped pipes). In instances where a perforated riser outlet is designed, the orifice is protected by the smaller perforations in the riser and a minimum orifice size of 50 mm is acceptable.

Length : Width Ratio

The flow path through a pond directly influences the overall performance. One of the most common problems associated with first generation pond designs was the construction of the outlet close to the inlet. Another common problem involved having multiple stormwater inlets at opposite ends of the pond based on stormwater servicing convenience.

All stormwater servicing should be conveyed to one inlet location at the pond. In order to provide the longest flow path through the pond the inlet to the pond should be located as far away as possible from the pond outlet. Recognizing that a specific storage volume is required to be provided in the pond the minimum flow path in a pond is generally described by the length to width ratio. A pond with a length to width ratio ≥ 3:1 will have an acceptable flowpath. Preferred length to width ratios range from 4:1 to 5:1.

An acceptable design feature to increase the length to width ratio is to provide berms in the pond to re-direct flows at certain elevations. This measure increases the pond performance by ensuring that short circuiting cannot occur. If the bermed areas are vegetated, the vegetation will help to filter the stormwater further enhancing the performance. In some areas of the province this is called a "serpentine" design. The addition of berms, however, will increase the land consumption of ponds.

Permanent Pool Depth

The average permanent pool depth in a wet pond should be 1 to 2 metres. The maximum depth in a wet pond should be restricted to 3 metres. Although ponds deeper than 3 metres may have some benefits in terms of temperature, deep ponds will become stratified and the reduced oxygen content may create anoxic conditions releasing metals and organics from the pond sediments.

Extended Detention Storage Depth

The extended detention storage depth should be limited to 1 to 1.5 metres. Although the wet pond consists of mostly open water areas, a planting strategy should be incorporated around the perimeter of the pond to enhance the performance of the pond and provide aesthetic and safety benefits. The depth restriction in the extended detention storage portion of the pond is related

to the planting strategy since some plant species cannot withstand water level fluctuations in excess of 1 -1.5 metres. The other reason for a depth restriction in the extended detention portion of the pond is to ensure that the overall depth of the pond (permanent pool and extended detention) is less than 5 metres.

Planting Strategy

A planting strategy is required for any wet facility to provide shading, aesthetics, safety, and enhanced pollutant removal. Native species should be used in all planting strategies. The planting strategy can be divided into five zones based on average water depth and soil moisture regime (frequency of wetting/inundation) :

1. deep water areas (submergent vegetation)
2. shallow water areas (emergent vegetation)
3. extended detention or shoreline fringe areas
4. floodfringe areas (if the facility is a combined quality/quantity facility)
5. upland areas

Deep Water Areas

Wet ponds are comprised of mostly deep water areas. Plantings in deep water areas are restricted to submergent vegetation. This type of vegetation is difficult to establish using seeds or rhizomes. Pondweeds can be planted in water depths of 2 m - 3 m. Other submerged species (listed in Appendix F) can be planted in water depths between 1 m - 2 m. Shallow and deep water plantings will become established according to water level fluctuations and critical depths for light availability. It is expected that there will be some intergradation of species between the 1 m and 3 m depths.

Shallow Water Areas

Shallow water areas represent areas where the permanent pool is ≤ 0.5 metres deep. In wet ponds, shallow water areas are usually restricted to the perimeter of the pond. The selection of vegetation in these areas should be based on achieving several objectives :

- nutrient uptake
- filtration of stormwater
- safety
- enhancing the aesthetics of the pond

Plantings in this zone provide ancillary benefits including :

- prevention of re-suspension of bottom sediments
- reduction of flow velocities promoting sedimentation

This zone includes both submergent and emergent vegetation. Typical species are listed in Appendix F. Submerged species should be planted starting at a water depth of 0.5 m from the shoreline. The majority of submerged species should be planted in water depths between 0.3 m and 0.5 m. Emergents should be planted starting at a water depth of 0.3 m. Species such as sedges, reed grass, and arrowhead should be planted at the water's edge. The side slopes constrain the amount of vegetation that can be established. A minimum slope is preferable in order to maximize the area available for plantings.

Shoreline Fringe Areas

Shoreline fringe areas represent the area which is subject to frequent wetting as a result of storm events. This area can be delineated by the land between the permanent pool and high water mark (extended detention storage) for erosion/water quality control. This zone will be subject to higher soil moisture conditions as a result of water level fluctuations during storm events and the influence of the permanent pool itself during dry weather conditions.

The objectives in the design of the planting strategy for the shoreline fringe area are similar to those in the shallow marsh area. The growing conditions in the fringe area, however, are harsher than the marsh area due to the frequent wetting/drying cycle which the plants must endure. Consequently, this area requires close attention during the design phase as well as implementation stage to ensure that the proposed plants become established.

Plant material in the shoreline fringe areas includes hardy hydric grasses and shrubs. A list of common species is provided in Appendix F. The grasses should be sown using a mixture in the spring or fall (spring is preferable) from the water's edge of the permanent pool to the limit of the extended detention. The shrubs should be planted in the extended detention zone such that only their lower branches will be inundated during the design storm. At least two shrub species should be planted to improve survival success.

Given the harsh growing conditions in this zone, it may take a considerable time to establish the vegetation, and replanting may be periodically required.

Floodfringe Areas

If the wet pond is used to control peak flow rates during infrequent storm events (2 year to 100 year), a zone of infrequent inundation will be created (floodfringe area). The influence of the permanent pool and frequent storm events is less pronounced for this area. The planting strategy in this zone includes a range of grass, herb, shrub, and tree species. A commercially available grass and herb seed mixture suitable for slope stabilization is recommended. Typical shrub and tree species are listed in Appendix F.

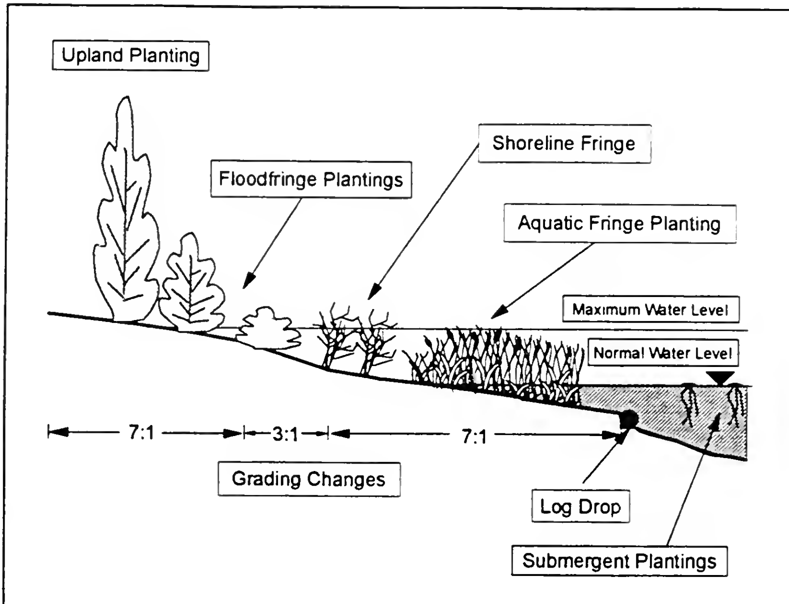


Figure 3.12 Wet Pond Grading and Planting Strategy

The grass and herb seed mixture should be sown in the spring or fall (spring is preferable). At least three species of shrubs and three species of trees should be planted in this zone. There should be a gradated change in planting near the upland zone to maximize the aesthetics in this area.

Upland Areas

Upland areas represent the landscaped areas provided as aesthetic amenities around the pond. Upland plantings should also be designed to restrict access to steep areas or inlet/outlet locations. A mixture of 5 plant species should be planted in a random pattern to prevent the establishment of monoculture areas. A list of typical species is presented in Appendix F. A large number of young plant stocks, tree whips and seedlings, should be implemented rather than a small number of large shrubs and caliper trees. Some caliper trees and mature plants should be used, however, to provide immediate wind screening, shading, aesthetic, and safety objectives. The selection of plant materials should consider :

- the topography and surface drainage
- soil conditions

- adjacent plant communities
- the potential for on-site transplantation
- the availability of nursery stocks

A naturalized landscape approach should be used which requires no maintenance and is sustainable over the long term. The upland plantings should provide a minimum of a 3 m buffer strip from the maximum design water level mark. The massing of trees and shrubs should be augmented by designated regeneration areas to achieve long term growth.

Fencing

The use of permanent fencing is left to the discretion of the local municipality. Fencing prevents the integration of the pond within a natural setting, however, and defeats the purpose of using natural system designs for water management. In areas where ponds adjoin the valley corridor, fencing will impede wildlife passage. Therefore, it is recommended that natural solutions be investigated (ie. grading and planting strategies noted in next section) to build in safety features for the pond.

Temporary fencing may be required after the initial construction period until the vegetation is established. In these situations post and wire fencing is recommended. The openings in the fence should be $\geq$ 150 mm $\times$ 150 mm. The temporary fencing should be removed once the vegetation in the planting strategy is established, and the cost of removal included in the development agreement.

Signs around the pond indicating the pond's purpose and function also help to inform the public of the potential for water level fluctuations in the pond during storm events.

Grading

The grading and landscaping plan near the pond edges is important to ensure public safety and to maximize the functionality of the pond.

Terraced grading (ie. a section of 7:1 grading, then 3:1 grading, then 7:1 grading) (Figure 3.12) is recommended to minimize the potential for the public to fall into the pond. Grading is critical at the permanent pool elevation. It is recommended that the land where the permanent pool meets the surrounding landscape land be graded at a minimum slope of 5:1 . This flat grading should extend a minimum of 3 metres on either side of the permanent pool elevation.

At the limit of the initial flat grading into the permanent pool, small drops (150 mm - 300 mm) can be incorporated using logs or stones to warn people who gain access to the water that the pond is becoming deeper.

Inlet Configuration

The stormwater conveyance system (sewers, grassed swales) should ideally have one discharge location into the wet pond. This requires planning and ongoing interaction between land use planners and municipal engineers to ensure that it is technically feasible and economically efficient to drain the tributary area to one inlet location. Multiple inlets, although undesirable may be required because of physical and economic constraints.

Exposed pilot channels (typically rock lined channels which convey stormwater from a pipe outlet to the pond) should be avoided since monitoring which has been performed indicates that these channels increase water temperature by 1°C for every 75 metres of pilot channel (Galli, 1990).

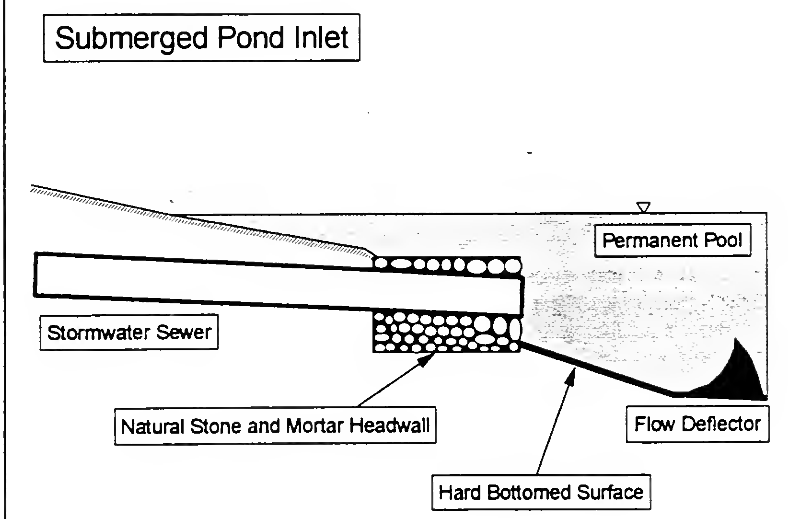


Figure 3.13 Submerged Pond Inlet

In areas where there is sufficient topographic relief, the inlet can be submerged below the permanent pool. This design has both advantages and drawbacks :

Advantages

- safety and vandalism (inlet is inaccessible)
- aesthetics

Drawbacks

- surcharging or backwater effect on the upstream stormwater conveyance system
- scour/re-suspension of the pond bottom near the inlet
- clogging of the inlet by sedimentation near the inlet
- maintenance implications of sediment deposition in the upstream conveyance system

A design which incorporates a submerged inlet requires a greater level of analysis at the design stage due to these potential drawbacks.

Submerged inlets should not be located at the bottom of the pond unless necessary. If the inlet is located at the pond bottom, the inlet design should incorporate a hard-bottomed surface near the inlet pipe to ensure that erosion and scour of the pond bottom does not occur. Other enhancements such as dissipation or deflection structures which direct flow away from the pond bottom also help to minimize scour and re-suspension of deposited sediment (Figure 3.13). Submerged inlets for piped systems with a flat grade (< 1%) should be avoided due to the potential for upstream surcharging (As a rule of thumb only the last 10 metres of pipe should be submerged near the discharge point).

Partially submerged inlets must be treated similarly to submerged inlets. The effect of the tailwater condition produced by a partially submerged, or submerged, inlet must be assessed with respect to upstream surcharging during minor system and major system events. Major system events are analysed to determine the requirements for inlet controls on catch-basins. The effect of partial or complete submergence is best evaluated using a dynamic hydraulic routing model such as EXTRAN. A conservative steady state analysis can be made, however, by assuming a constant tailwater elevation equivalent to the maximum design water level in the pond and assessing the upstream surcharge for the peak design event flow.

Unsubmerged inlets are generally easier to design since they do not introduce hydraulic complications into the system. Given the maintenance and re-suspension concerns, unsubmerged inlets are generally preferred over submerged inlets. The invert of the inlet pipe is set at the maximum design water level in the pond (assuming no flow splitter upstream, flow splitters are discussed in Section 3.5).

As a result of the raised inlet invert and pond side slopes, the unsubmerged inlet will not discharge directly into water. It is important that any erosion potential between the inlet and the permanent pool be addressed in this design.

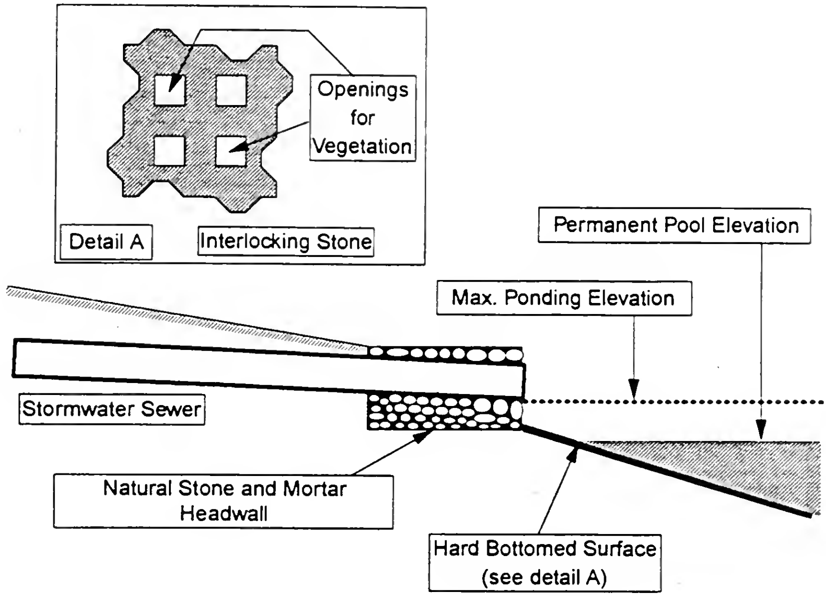


Figure 3.14 Unsubmerged Pond Inlet and Hard Bottom Forebay

The use of environmental stone/blocks (interlocking blocks with large openings to allow vegetative growth in between the blocks) in this area is recommended (Figure 3.14) in this regard since they will minimize the erosion potential and minimize water temperature increases.

It is important that the inlet area is deep (> 1 metre) to minimize concerns for re-suspension of settled pollutants. Guidance is provided in the following section on sediment forebays with respect to their design.

Outlet Configuration

The outlet should be located in the pond embankment for ease of operations, maintenance, and aesthetics. There are two main designs which are currently accepted for the drawdown of the quality/erosion portion of the pond :

- a reverse sloped outlet pipe
- a perforated riser outlet pipe

Reverse Sloped Pipe

A reverse sloped pipe (Figure 3.15) is appropriate for ponds with outlet areas ≥ 1 m deep. The reverse sloped pipe is used as the outlet in the water quality/erosion portion of the pond. The reverse sloped pipe should drain to an outlet chamber located in the pond embankment. The outlet chamber can contain openings for flood control detention and overflow protection. It is recommended that a gate valve be attached to the reverse sloped pipe in the outlet chamber. This valve will allow the extended detention drawdown time to be modified to improve pollutant removal if the pond is found to be operating outside of the design criteria.

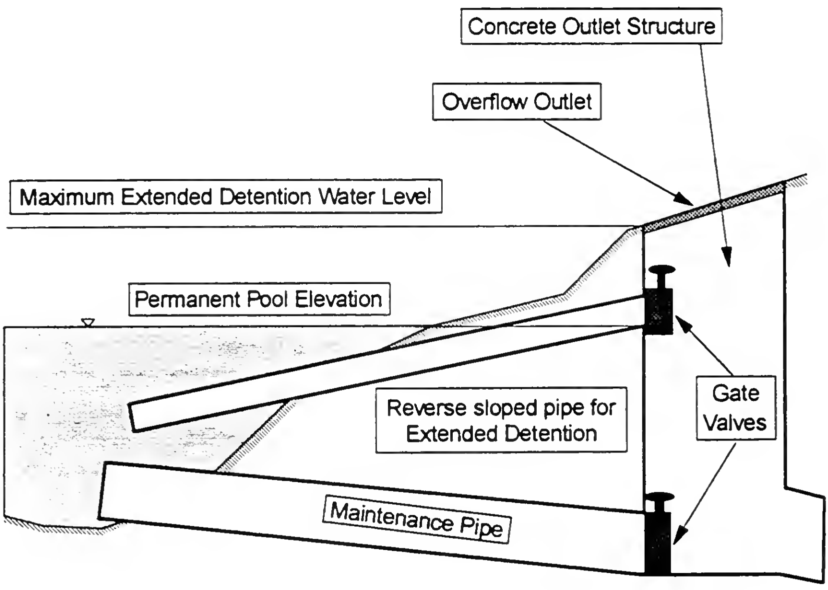


Figure 3.15 Reverse Sloped Pipe Outlet Configuration

A low flow maintenance pipe should be provided to drain the pond for maintenance purposes. The maintenance pipe should also drain to the outlet chamber. It is recommended that the maintenance pipe be sized to provide a 6 hour detention time (6 hours was chosen as a reasonable time period in which to drain the entire pond for maintenance recognizing that the release rate should not affect the downstream receiving waters), and that a gate valve be attached to the end of this pipe in the outlet chamber.

Perforated Riser Pipe

A perforated riser pipe is the traditional outlet pipe that has been used throughout Ontario. The riser itself is perforated with holes. Typical hole diameters range from 12 mm to 25 mm.

The flow through the riser is controlled by an orifice plate located at the bottom of the riser structure. The smallest orifice diameter which should be used is 50 mm.

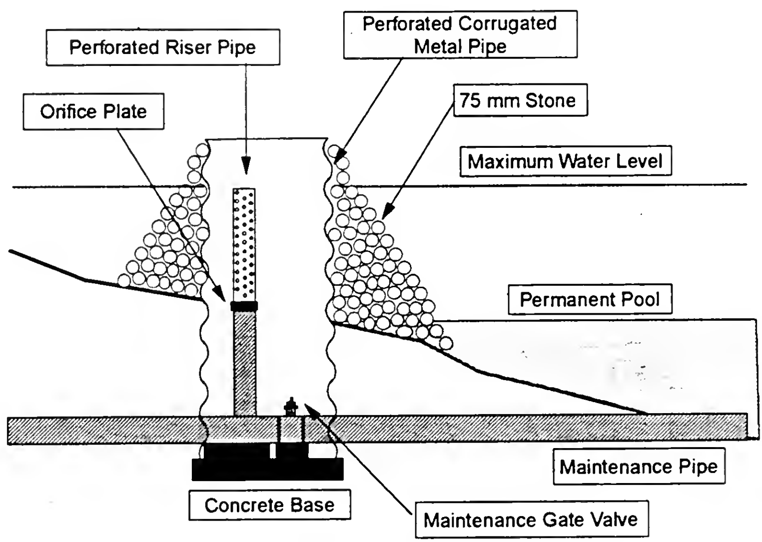


Figure 3.16 Perforated Riser Pipe Pond Outlet Configuration

A typical perforated riser outlet structure design which is frequently used in Ontario incorporates a perforated riser pipe surrounded by a corrugated metal pipe standing on its end. Holes (50 mm diameter) are drilled in the metal pipe such that it acts as a riser. Stone is placed around the metal pipe (minimum 75 mm diameter) to act as a further filter. This design is shown in Figure 3.16.

Although this design is inexpensive, and should be resilient to clogging by suspended solids, there are several drawbacks which the engineer should keep in mind if this design is chosen :

- if the structure is not located in a chamber in the embankment it will have to be located in the pond itself. This type of outlet will look unnatural and will be

aesthetically unappealing

- corrugated metal pipe which has holes drilled in it will rust resulting in a shorter life span compared to other materials
- since the riser is above the permanent pool it will be more susceptible to clogging by trash

A similar design of a perforated riser pipe in an outlet structure in the embankment (to address aesthetics and maintenance access) is provided in Figure 3.17.

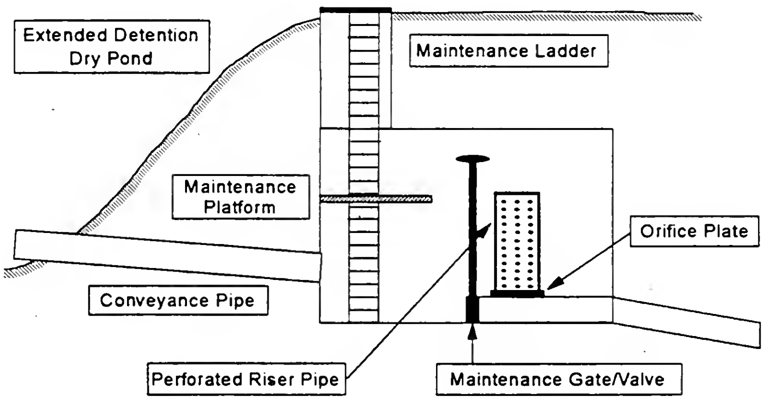


Figure 3.17 Perforated Riser Outlet in Embankment

Water can be conveyed to the chamber by either a positively sloped (> 0.5%) pipe or a reverse sloped pipe (> 0.5%). If a positively sloped conveyance pipe is used it should be larger than 250 mm diameter to minimize the risk of clogging.

The riser itself and fittings should be constructed of a durable plastic or similar material. Holes should be drilled into the riser (13 mm - 25 mm diameter) along the entire length of the riser. The diameter of the pipe, and hence the number of openings, should be sufficient to ensure that the openings do not provide the extended detention control. The riser should be connected to the outlet pipe discharging from the chamber.

In the event that the riser does clog, there should be a maintenance gate or valve in the outlet chamber. A by-pass pipe which routes flow directly to the outlet pipe around the chamber is

preferable, but is also more expensive. An illustration of this outlet system is shown in Figure 3.17.

Although the perforated riser outlet design has been used for wet ponds it is best suited to ponds with a shallow permanent pool (ie. wetlands) or ponds without a permanent pool (extended detention dry ponds).

Outlet Channel

Most ponds which discharge directly to a stream will be in close proximity to the receiving waters (the pond will likely be located in the lowest section of the tableland adjacent to the receiving waters). The outlet channel in most of these cases will be short. In cases where the outlet channel is lengthy, (ie. traverses a wide floodplain) natural channel design techniques should be used to ensure that the channel conforms to the natural characteristics of the valleylands. Guidance on natural channel design techniques is provided in "Natural Channel Systems : An Approach to Management and Design, Development Draft" (Ministry of Natural Resources, 1994).

Temperature Control

Wet Ponds can have a negative effect on receiving water temperatures. Guidance on measures to ensure that the receiving water temperatures will not be significantly altered are provided in Section 3.9.

Maintenance and Performance Enhancements

There are several guidelines which will improve the performance of a wet pond with respect to pollutant removal and facilitate maintenance activities :

- sediment forebay
- hard bottomed access for maintenance
- low flow berming

Sediment Forebay

A sediment forebay facilitates maintenance and improves pollutant removal by trapping larger particles near the inlet of the pond. The forebay should be one of the deeper areas of the pond to minimize the potential for re-suspension and to prevent the conveyance of re-suspended material to the pond outlet.

The forebay sizing depends on the inlet configuration, and several calculations can be made to ensure that it is adequately sized.

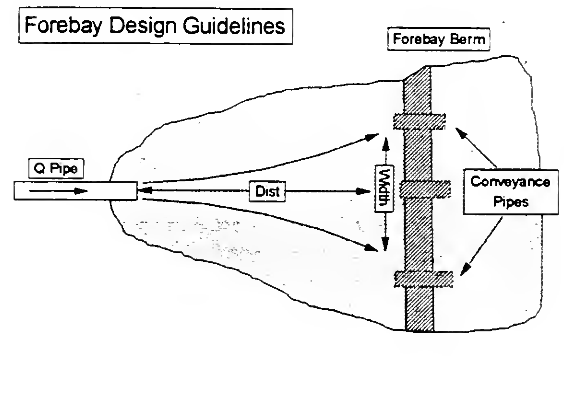


Figure 3.18 Wet Pond Forebay

1) Settling Calculations

The primary method to calculate the forebay volume and length should be based on settling calculations. These calculations involve determining the distance to settle out a certain size of sediment in the forebay. The methodology assumes that the flow out of the pond dictates the velocity through the forebay and rest of the pond.

Although this is not strictly correct, it is reasonable for the determination of an appropriate forebay length.

The settling velocities for different sized particles can be estimated from the stormwater particle size distribution monitoring data (U.S. EPA, Table 3.3) which was collected in the NURP (Nationwide Urban Runoff Program, 1983) study. It should be noted that the settling velocities provided in Table 3.3 are much lower than those calculated by Stokes' Law or Newton's Law. These latter laws, however, are for ideal settling characteristics and spherical particles, neither or which occur in the field.

Table 3.3 Particle Size Distribution in Storm Water		
Size Fraction	% of Particle Mass	Average v_s (m/s) ≤
≤ 20 μm	0 - 20	0.00000254
20 μm ≤ x ≤ 40 μm	20 - 30	0.0000130
40 μm < x ≤ 60 μm	30 - 40	0.00002540
60 μm < x ≤ 0.13 mm	40 - 60	0.00012700
0.13 mm < x ≤ 0.40 mm	60 - 80	0.00059267
0.40 mm < x ≤ 4.00 mm	80 - 100	0.00550333

Equation 3.3 defines the appropriate forebay length for a given settling velocity and hence particle size to be trapped in the forebay. A derivation of Equation 3.3 is provided in Appendix G.

$$Dist=\sqrt{\frac{rQ_p}{V_s}}$$

Equation 3.3 Forebay Settling Length

where Dist= Forebay length (m)
r = Length to width ratio of forebay
Q_p= Peak flowrate from the pond during design quality storm
V_s= Settling velocity (dependent on desired particle size to settle)

A forebay design to settle out particles smaller than 100μm, given the settling velocities shown in Table 3.3, requires the pond to be greatly oversized compared to normal standards. Therefore, it is generally recommended that the forebay be designed to settle out 150 μm particles. The settling velocity for 150 μm particles based on Table 3.3 is estimated to be 0.0003 m/s (which is approximately one order of magnitude less than the settling velocity for 40 μm given by Stokes' Law).

In all instances the forebay should not exceed one third of the pond surface area. In addition, the length to width ratio in the forebay itself should be ≥ 2:1.

2) Dispersion length

The dispersion length refers to the length of fluid required to slow a jet discharge (ie. pipe flow). A check can be made on the forebay length given by the settling calculation (Equation 3.3) to ensure that there is adequate dispersion. Equation 3.4 provides a simple guideline for the length of dispersion required to dissipate flows from the inlet pipe. It is recommended that the forebay length is such that a fluid jet will disperse to a velocity of ≤ 0.5 metre/second at the forebay berm.

The dispersion length is usually smaller than the settling length unless there is a large upstream urban drainage area (eg. 100 ha) or the pond is subject to large inflows (ie. a combined quantity and quality facility). In cases where a combined facility is designed, the dispersion length should be calculated for the pipe design capacity unless the pipe is designed for storms larger than a 10 year return period. In cases where the pipe conveys flows in excess of a 10 year storm, the dispersion length should be calculated for 10 year flows. In all cases, the forebay length should be greater than, or equal to, the larger of the two forebay lengths given by the Equation 3.3 and Equation 3.4.

$$\text{Dist} = \frac{8Q}{dV_f}$$

Equation 3.4 Dispersion Length

where
- Dist = length of dispersion (m)
- Q = inlet flowrate (m^3/s)
- d = depth of the permanent pool in the forebay (m)
- V_f = desired velocity in the forebay

The depth of the permanent pool in the forebay in Equation 3.4 reflects the deep section (> 1 m) of the forebay required to minimize re-suspension and scour. A guideline for the minimum bottom width of this deep zone is given by :

$$\text{Width} = \frac{\text{Dist}}{8}$$

Equation 3.5 Minimum Forebay Bottom Width

Generally, the total width of the forebay should provide a length to width ratio ≥ 2:1 if a single inlet is proposed for the pond. A length to width ratio < 2:1 is undesirable since the storage will not be effectively utilized in this configuration (dead zones).

Although Equation 3.4 provides the length of forebay to ensure a certain velocity in the discharge jet at the end of the forebay, a check should be made using the entire forebay cross-sectional area to ensure that the average velocity in the forebay is less than, or equal to, 0.15 m/s. This velocity (0.15 m/s) is noted empirically (Figure 4.6) as the maximum permissible velocity before which erosion will occur in a channel.

The design flowrate in Equation 3.4 is the peak flowrate of the water quality storm. If this value is not known (eg. the subwatershed plan specifies the pond sizing based on continuous simulation) it can be approximated using either standard design event modelling practices with a 4 hour Chicago distribution of a 25 mm storm, or using the Rational Method (Equation 3.6) with an intensity given by Equation 3.7. A derivation of Equation 3.7 is provided in Appendix H.

$$Q = \frac{C\,i\,A}{360}$$ Equation 3.6 Rational Method

where
Q = peak flow rate (m^3/s)
C = runoff coefficient
i = rainfall intensity (mm/h)
A = drainage area (ha)

$$i = 43C + 5.9$$ Equation 3.7 25 mm Storm Intensity

where
i = rainfall intensity (mm/h)
C = runoff coefficient

3) Cleanout Frequency

A check on the permanent pool volume contained in the forebay can be made by estimating the accumulation of sediments in the forebay. A conservative estimate would be to assume the maximum facility removal efficiency (given in Figures 4.2 through 4.5) in the forebay and to ensure that the forebay volume is equal to, or greater than, 10 years of sediment accumulation. Values of sediment loading/accumulation per hectare of contributing drainage area are provided in Section 5.5 (Table 5.3) based on the upstream catchment imperviousness.

Forebay Berm

The forebay should be separated from the rest of the pond by an earthen berm. The berm can be submerged slightly below the permanent pool or it can extend into the extended detention portion of the pond. Pipes can be installed in the berm as either the primary conveyance system from the forebay to the pond, or as a secondary conveyance system to supplement flows over a submerged berm. In either case flow calculations should be made to ensure that the berm does not provide a flow restriction which would cause the entire forebay (not just the berm) to overflow under design conditions.

The inverts of any conveyance pipes installed in the berm should be set at least 0.6 m above the bottom of the forebay. This will prevent the siphoning of settled material from the bottom of the forebay into the rest of the pond. A maintenance pipe should also be installed in the berm to drawdown the forebay for maintenance purposes. If only the forebay is drawn down during maintenance (ie. maintenance pipe connects to the outlet directly and/or the forebay will be pumped out) the forebay berm must be designed as a small dam since the rest of the pond will not be drained.

If a submerged berm is implemented, the berm height should be 0.15 - 0.30 metres below the permanent pool elevation. A submerged berm provides additional safety benefits (the public is not tempted to walk on the berm) and allows wetland type plantings around the berm. The berm should be planted with emergent vegetation to promote filtration of water as it passes over the berm. Suitable species of plants include fragrant waterlily, american bulrush, and softstem bulrush. The plants should be established on the top and sides of the berm at a maximum planting depth of 30 cm.

Hard Bottomed Surface

Experience in Ottawa with wet ponds indicated many operational and maintenance problems (Gietz, 1983). One of the problems was removing sediment which accumulated in the ponds. Although the ponds were drawn down to allow vehicular access for maintenance, natural pond bottoms proved to be too soft and hindered maintenance. In order to facilitate maintenance it is recommended that pond forebays be lined with a hard-bottomed surface. The surface should have openings to allow the growth of plant material, but also be capable of withstanding the weight of grading equipment which will be used to maintain the pond. Several manufacturers currently distribute this type of product in Ontario. The hard bottomed surface should extend to the vegetated shoreline fringe in the forebay area. One or two access routes into the pond should be constructed using the hard bottom surface.

A hard-bottomed surface can also be installed near the outlet of the pond to provide an access route to the outlet pipes for maintenance. Although desirable, this treatment is less important than providing a hard-bottomed access route near the inlet of the pond.

Low Flow Berming

A common problem in pond design is an imperfect pond shape/configuration as a result of the existing topography and/or approved land uses. In these instances, low flow berming (Figure 3.19) can be designed to help extend the flow path in the pond.

The berms extend from the pond bottom to the extended detention area of the pond. Generally the berms extend to within 150 - 300 mm of the permanent pool elevation. The number and orientation of the berms is based on maximizing the flow length in the pond. It should be recognized that berming (topographical changes) consume volume in a pond, and that additional area is required to maintain the treatment volume when berming is used.

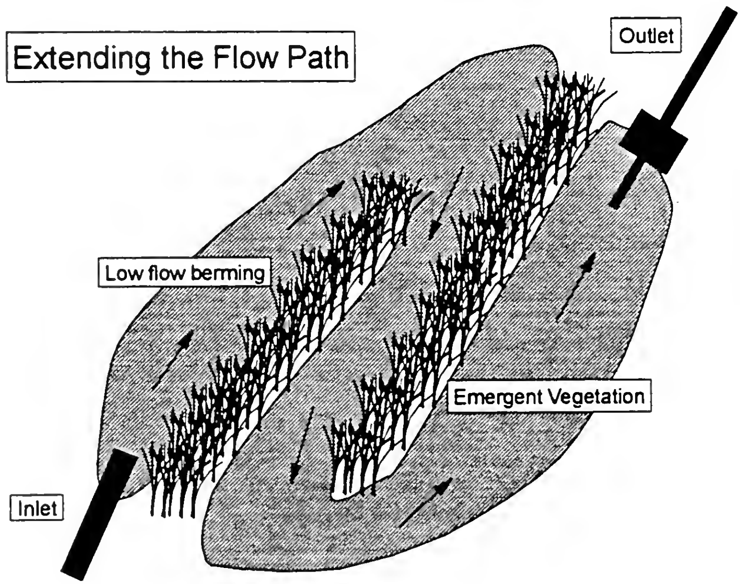


Figure 3.19 Low Flow Berming

Emergent vegetation should be planted on the berms to help redirect flows and filter stormwater. The vegetation also acts as a deterrent to prevent people from walking on the bermed areas.

3.4.4 Dry Ponds

Extended detention dry ponds have no permanent pool of water. As such, the removal of stormwater contaminants in these facilities is purely a function of the drawdown time in the pond. Re-suspension is a concern in dry ponds due to the lack of a permanent pool, especially if their intended function includes quantity control (ie. a flow splitter is not included as part of the design).

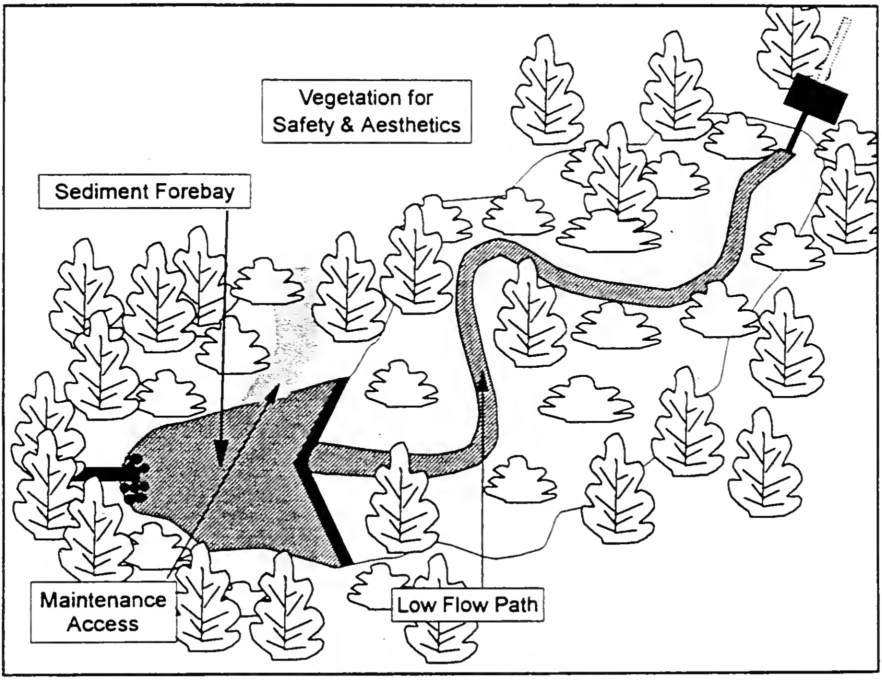


Figure 3.20 Extended Detention Dry Pond with Forebay

Design Guidance

Common sense guidelines have the greatest impact on the performance of a dry pond. These guidelines can be classified into several groups :

- drainage area
- length to width ratio
- planting strategy

- grading
- flow splitter (Section 3.5)
- inlet and outlet configurations
- maintenance and performance enhancements

Storage Sizing

In areas with subwatershed planning, guidance will be provided with respect to the active storage (extended detention) sizing.

If a subwatershed plan has not been, and will not be undertaken, guidance for the sizing of the active storage is provided in Chapter 4. The guidance on sizing provided in Chapter 4 is based on long term suspended solids removal using continuous simulation (Appendix I).

Drainage Area

As a general rule dry ponds should be implemented for drainage areas ≥ 5 hectares. This area requirement is purely a function of the outlet sizing to ensure that the outlet does not become clogged. This drainage area restriction does not apply in cases where real time control is implemented or batch control is used.

Detention Time

A detention time of 24 hours should be targeted in all instances, unless the potential for clogging the outlet is high.

In cases where the outlet is susceptible to clogging (ie. drainage areas < 8 ha - see minimum orifice size) the detention time can be reduced to a minimum of 12 hours. (Note : simulations that were performed in this study indicated that a 12 hour detention time produced similar settling to a 24 hour detention time)

The drawdown time in the pond can be estimated using Equation 3.2. Equation 3.2 is the classic falling head orifice equation which assumes a constant pond surface area. Although this is not strictly true, the calculation can be done in steps corresponding to different pond areas or the average surface area can be estimated. A discussion of this equation is provided in Appendix E.

Minimum Orifice Size

The smallest diameter orifice accepted by most municipalities to ensure that clogging does not occur in a stormwater system is 75 mm. It is recommended that this minimum size be maintained for exposed outlet designs (ie. reverse sloped pipes). In instances where a perforated riser outlet is designed, the orifice is protected by the smaller perforations in the riser and a minimum orifice size of 50 mm is acceptable.

Length : Width Ratio

The flow path through a pond is one of the main determinants of its performance. One of the most common problems associated with first generation pond designs was the construction of the outlet close to the inlet. Another common problem involved having multiple stormwater inlets at opposite ends of the pond since it was convenient from a stormwater servicing perspective.

All stormwater servicing should be conveyed to one inlet location at the pond if possible. In order to provide the longest flow path through the pond, the inlet to the pond should be located as far away from the pond outlet as possible. The desired flow path in a pond is generally described by the length to width ratio recognizing that a specific storage volume is to be provided in the pond . A pond with a length to width ratio ≥ 3:1 will have an acceptable flowpath. Preferred length to width ratios range from 4:1 to 5:1.

An acceptable design feature to increase the length to width ratio is berming to re-direct flows at certain elevations. This practice increases the pond performance by ensuring that short circuiting cannot occur. One drawback associated with berming within the pond is that additional land will be required to fulfill the storage requirements for water quality, erosion, and/or quantity control.

Extended Detention Storage Depth

The extended detention storage depth should be limited to 2 to 3 metres. This maximum applies to all extended detention objectives (ie. water quality, erosion, and quantity control). If a planting strategy is proposed for the dry pond, these maximum depths may be reduced to 1 to 1.5 metres. It is anticipated, however, that the dry pond will not be actively planted in the extended detention portion of the pond since harsh growing conditions (frequent wetting/drying) would hinder the establishment of an aggressive planting strategy.

Planting Strategy

The planting strategy for a dry pond is less aggressive (ie. fewer species and reduced planting intensity) than that for a wet pond. Plantings can be divided into three zones based on the soil moisture regime (frequency of wetting/inundation):

1. extended detention or shoreline fringe areas
2. floodfringe areas (if the facility is a combined quality/quantity SWMP)
3. upland areas

As with all SWMPs, native species should be used wherever available.

Extended detention Areas

Extended detention areas represent the area which is subject to frequent wetting as a result of storm events. This area can be delineated by the land between the pond bottom and the high water mark (extended detention storage) for erosion/water quality control. This land will be subject to fluctuating soil moisture conditions as a result of water level fluctuations during storm events and inter-event (dry weather) periods.

The growing conditions in the extended detention area for a dry pond are harsher than the same area in a wet pond since there is not the influence of a permanent pool. Consequently, this area requires close attention during the design phase as well as the implementation stage, to ensure that the proposed plants become established.

Plant material in the shoreline fringe areas include hardy hydric grasses and shrubs. A list of common species is provided in Appendix F. Grasses should be sown using a mixture in the spring or fall (spring is preferable) from the water's edge of the permanent pool to the limit of the extended detention. Shrubs should be planted in the extended detention zone such that only their lower branches will be inundated during the design storm. At least two shrub species should be planted to improve survival success.

Given the harsh growing conditions in this zone, it may take a considerable time to establish the vegetation, and replanting may be periodically required.

Floodfringe Areas

If the dry pond is also used to control peak flow rates during infrequent storm events (2 year to 100 year), a zone of infrequent inundation will be created (floodfringe area). The influence of the frequent storm events is less pronounced for this area. As such, plants in this area must be able to tolerate infrequent inundation for several hours. The planting strategy in this zone includes a range of grass, herb, shrub, and tree species. A commercially available grass and herb seed mixture suitable for slope stabilization is recommended. Typical shrub and tree species are listed in Appendix F.

The grass and herb seed mixture should be sown in the spring or fall (spring is preferable). At least three species of shrubs and three species of trees should be planted in this zone. There should be a gradated change in planting near the upland zone to maximize the aesthetics in this area.

Upland Areas

Upland areas represent the landscaped areas provided as aesthetic amenities around the pond. The upland plantings should provide a minimum of a 3 m wide buffer strip above the maximum design water level mark. Although the plantings in this area are mainly a function of aesthetics, other objectives to support the function of the pond should be

remembered such as :

- providing a wind barrier to prevent turbulence in the pond
- restricting access to steep areas or inlet/outlet locations
- shading to mitigate increases in water temperature due to ponding

A mixture of 5 shrub and tree species should be planted in a random pattern to prevent the establishment of monoculture areas. A list of typical species is presented in Appendix F. A large number of young plant stocks, tree whips and seedlings, should be implemented rather than a small number of large shrubs and caliper trees. Some caliper trees and mature plants should be used, however, to provide immediate wind screening, shading, aesthetic, and safety objectives. The selection of plant materials should consider :

- the topography and surface drainage
- soil conditions
- adjacent plant communities
- the potential for on-site transplantation
- the availability of nursery stocks

A naturalized landscape approach should be used which requires no maintenance and is sustainable over the long term. The massing of trees and shrubs should be augmented by designated regeneration areas to achieve long term growth.

Fencing

The use of permanent fencing is left to the discretion of the local municipality. Fencing prevents the integration of the pond within a natural setting, however, and defeats the purpose of using natural system designs for water management. In areas where ponds adjoin the valley corridor, fencing will impede wildlife passage. Therefore, it is recommended that natural solutions be investigated (ie. grading and planting strategies noted in next section) to build in safety features for the pond.

Temporary fencing may be required after the initial construction period until the vegetation is established. In these situations post and wire fencing is recommended. The openings in the fence should be $\geq$ 150 mm $\times$ 150 mm. The temporary fencing should be removed once the vegetation in the planting strategy is established, and the cost of removal included in the development agreement.

Signs around the pond indicating the pond's purpose and function also help to inform the public of the potential for water level fluctuations during storm events.

Grading

The grading in a dry pond is less critical than a wet pond since there is no permanent pool. The typical extended detention time of 24 hours, however, indicates that water will be present in these facilities for a reasonable period of time. In recognition of this, the grading of the pond side slopes should be terraced with an average slope of 4:1 or flatter.

Inlet Configuration

The stormwater conveyance system (sewers, grassed swales) should minimize the number of discharge locations into the pond. This requires planning and ongoing interaction between land use planners, municipal engineers, and stormwater management professionals.

The invert of the inlet pipe is set at the maximum design water level in the pond (assuming no flow splitter upstream, flow splitters are discussed in Section 3.5).

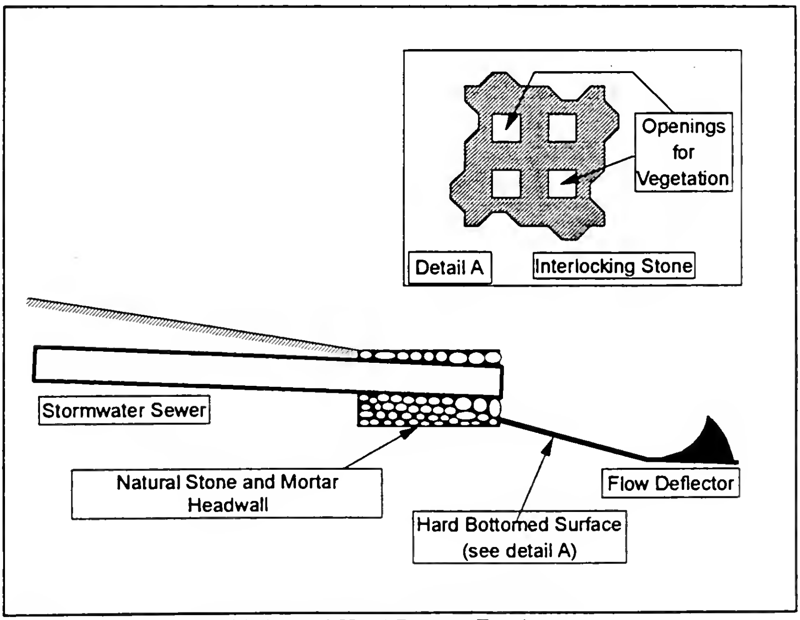


Figure 3.21 Dry Pond Inlet and Hard Bottom Forebay

As a result of the raised inlet invert and pond side slopes, the unsubmerged inlet will not discharge directly into water. It is important that the erosion potential is addressed in this design. The use of environmental stone/blocks (interlocking blocks with large openings to allow vegetative growth in between the blocks) in this area is recommended (Figure 3.21) in this regard since they will minimize the erosion potential.

A flow deflector or energy dissipation blocks can be used to reduce the potential for scour of previously settled pollutants from the pond bottom.

Outlet Configuration

There are numerous outlet configurations possible for dry ponds. The outlet should be located in the pond embankment wherever possible for ease of maintenance and aesthetics. Two configurations are presented to illustrate typical designs.

Perforated Riser

A perforated riser pipe can be used as the outlet in the extended detention portion of the pond. As indicated in Section 3.4.3, a perforated riser surrounded by a perforated corrugated metal pipe and 75 mm diameter stone can used. This outlet design is shown in Figure 3.16.

Reverse-sloped Pipe

A second type of outlet configuration utilizes a reverse sloped pipe. In this design a section of the pond near the outlet will not be gravity drained due to the reverse slope of the pipe. A portable pump must be used to drain this portion of the pond for maintenance. The use of under-drains (ie. tile drains) under ponds has not been historically effective in draining these facilities and is not recommended.

Experience with reverse sloped pipes indicates that they are resilient to clogging (Schueler, 1992). As such, the riser pipe and orifice connection are replaced by a gate valve or sluice gate on the reverse sloped pipe at the outlet chamber. If a valve is used, a gate valve is preferable to a globe valve given the size of valve required, and hence cost. This gate/valve will allow the manipulation of the outlet to achieve the desired settling characteristics in the field.

In addition, since the detention control is located on the inlet pipe to the chamber, rather than the outlet pipe from the chamber, the chamber can be used as a flood control outlet or emergency overflow outlet. For example, the inlet chamber can have a grated top, or weir openings along its side adjacent to the pond to provide further water management control.

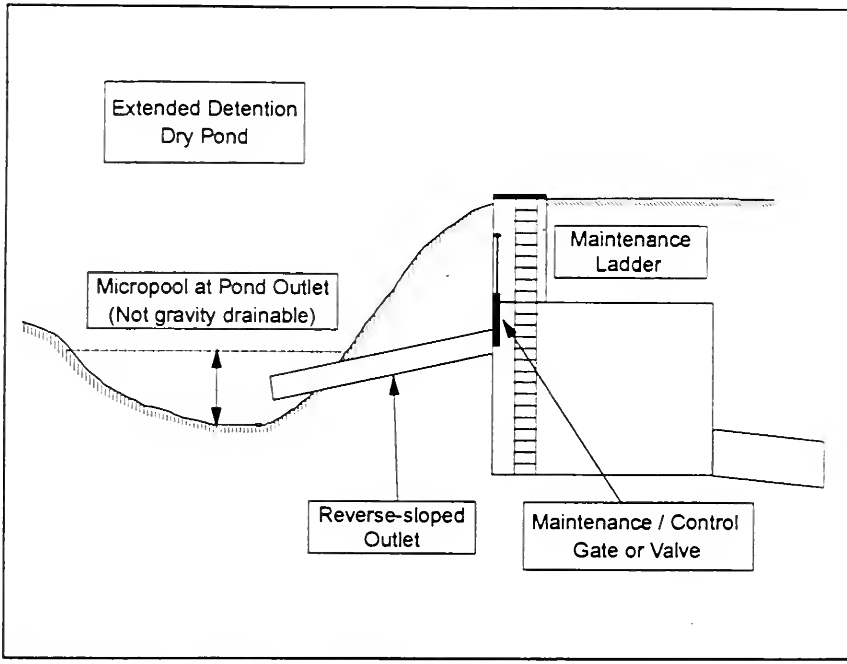


Figure 3.22 Dry Pond Reverse-sloped Outlet Pipe

Maintenance and Performance Enhancements

There are several features which will improve the performance of a dry pond with respect to pollutant removal and facilitate maintenance activities :

- sediment forebay
- batch operation

Sediment Forebay

A sediment forebay facilitates maintenance and improves pollutant removal by trapping larger particles near the inlet of the pond. The forebay should include a deep permanent pool (> 1 metre) to minimize the potential for scour and re-suspension. The forebay sizing depends on the inlet configuration, and several methods for sizing are provided in Section 3.4.3 (Wet Ponds).

Forebay Berm

The forebay should be separated from the rest of the pond by an earthen berm. The berm should be designed as a small dam since the downstream section of the pond will be dry. A weir should be designed at the top of the berm to convey flows to the downstream section of the pond during storm events. A maintenance pipe should be installed in the berm to allow the forebay to be drawn down during cleanout. This pipe would be opened and closed by a valve located on the upstream end of the pipe. Under normal operation the valve would be closed such that the only means of conveyance would be the weir flow over the forebay berm. Flow calculations should be made to ensure that the berm does not provide a flow restriction which would cause the entire forebay to overflow (not just over the berm) under design conditions because the berm does not provide adequate conveyance capacity. During maintenance periods, the valve would be opened allowing the forebay to be drained.

The berm should be planted with emergent vegetation to promote filtration of water as it passes over the berm. Suitable species of plants include american bulrush and softstem bulrush. The plants should be established on the forebay side of the berm at a maximum planting depth of 30 cm.

Pond Forebay Bottom

Experience in Ottawa with wet ponds indicated many operational and maintenance problems (Gietz, 1983). One of the problems was removing sediment which accumulated in the ponds. Although the ponds were drawn down to allow vehicular access for maintenance, natural pond bottoms proved to be too soft and hindered maintenance. In order to facilitate maintenance it is recommended that pond forebays be lined with a hard-bottomed surface. The surface should have openings to allow the growth of plant material, but also be capable of withstanding the weight of grading equipment which will be used to maintain the pond. Several manufacturers currently distribute this type of product in Ontario. The hard bottomed surface should extend to the vegetated shoreline fringe in the forebay area. One or two access routes into the pond should be constructed using the hard bottom surface.

Batch Control Operation

Continuous simulation of stormwater management facilities for urban development was performed as part of this study (Appendix I). These simulations investigated two modes of operation for dry ponds : continuous and batch. Continuous operation utilizes a simple hydraulic outlet such as a perforated riser or reverse-sloped pipe. Batch mode operation requires real time control of the facility using electrical and mechanical equipment.

Most municipalities discourage or forbid the use of electrically driven real-time controls for stormwater management since they are perceived as unnecessary operational and maintenance liabilities. The necessary components of a simple batch control operation can be described as follows :

- flow and/or water level sensors or rain gauge to indicate when a storm is occurring for the closure of the pond outlet
- a datalogger/processor which records the rain/stormwater information and operates the outlet control
- a sluice gate outlet which is raised or lowered using an electric motor controlled by a timer delay
- electrical power to operate the outlet, sensing equipment, and datalogger/processor control system

Using this equipment, the outlet would be closed when sufficient flow/water level or rainfall was recorded. At the same time a timer would be activated to record the retention time (eg. 24 hours). The outlet gate would be opened once the timer reached the preset retention time.

Batch control was tested for stormwater control in Ottawa by the Regional Municipality of Ottawa Carleton (RMOC) during the 1970s. Their monitoring indicated that batch operation of stormwater facilities resulted in higher performance (pollutant removal) compared to continuous operation (Gietz, 1983). This finding agrees with the results of the continuous modelling undertaken in this study. The modelling indicated that batch operation of dry ponds would result in a long term increase in pollutant removal of approximately 20% compared to continuous operation (24 hour hydraulic drawdown) for the same storage volume.

Recently, the City of Nepean constructed a large wet pond with an ultra-violet disinfection unit which utilizes real-time control. Generally, however, real time control is implemented as an exception rather than a rule. In cases where batch control is implemented there must be stringent inspection/maintenance requirements due to the potential for problems (blocked outlet structures, inaccurate sensing equipment, motor failures, etc.).

Technical Effectiveness

Dry ponds that operate in a continuous mode are less effective than wet ponds for pollutant removal (Chapter 4). Dry ponds that operate in a batch mode have a comparable effectiveness with wet ponds but have greater long term operational and maintenance requirements/costs (electrical power, inspections, calibration, replacement of worn electrical/mechanical equipment, etc.).

In general, dry ponds should only be implemented if it is determined that a wet pond or wetland cannot be implemented (ie. temperature concerns, land constraints). Although the performance of dry ponds operating in a batch mode is similar to wet ponds, real time control of stormwater ponds should be the exception rather than the rule. Batch operation entails higher long term operational and maintenance costs and detracts from the natural systems approach to stormwater management control.

Combined quality and quantity control in dry ponds is acceptable as long as a sediment forebay

(which includes a permanent pool) is incorporated in the design. Combined quality and quantity control is discouraged in dry ponds without forebays due to the potential for scour and re-suspension of previously settled pollutants.

3.4.5 Constructed/Artificial Wetlands

The constructed/artificial wetland is one of the most promising end-of-pipe SWM facilities for water quality enhancement. The benefits of constructed wetlands are similar to wet ponds and include :

- the performance does not depend on soil characteristics
- the permanent pool minimizes re-suspension
- the permanent pool minimizes blockage of the outlet
- the biological removal of pollutants (enhanced nutrient removal)
- the permanent pool provides extended settling

Artificial wetlands also have similar environmental impacts to wet ponds related to increased downstream water temperature which may limit their application in certain areas (see Section 3.9).

Constructed, or artificial, wetlands are the least understood end-of-pipe SWM facilities in terms of their biological impacts and enhancements. For example, wetlands will provide ancillary aviary, terrestrial, and aquatic habitat. There is a reluctance to design wetlands specifically for these objectives, however, since concerns have been raised relating to bio-accumulation of stormwater contaminants, and the destruction of habitat during maintenance of the wetland.

The controversy surrounding wetlands centres on the tendency to consider stormwater wetlands as natural wetland systems. The loss of natural wetlands is, in itself, a major issue which has led to a provincial wetland policy. There are those who think that the implementation of stormwater wetlands will be a replacement for the loss of natural wetlands. While stormwater wetlands will provide water quality enhancement similar to natural wetlands, they will not have the same attributes as natural wetlands. **It must be recognized that stormwater wetlands are first and foremost stormwater management facilities that must be maintained.** Stormwater wetlands should never be considered as significant natural areas which require environmental protection. Similarly, **the use of natural wetlands for stormwater quality enhancement is not allowed** since the introduction of stormwater could alter the hydrologic regime and chemical/biological composition of the wetland.

Wetlands have been noted to accumulate total phosphorus, but export ortho-phosphorus (the phosphorus which results in algae blooms) and metals such as zinc during the fall as the wetland plants begin to decompose (Novotny, 1984; Martin, 1988; Bayley, 1986). These findings have given rise to the harvesting of wetland plant material to prevent the export of pollutants, while others argue that the release of contaminants (namely phosphorus) during the fall has a negligible

impact on downstream resources.

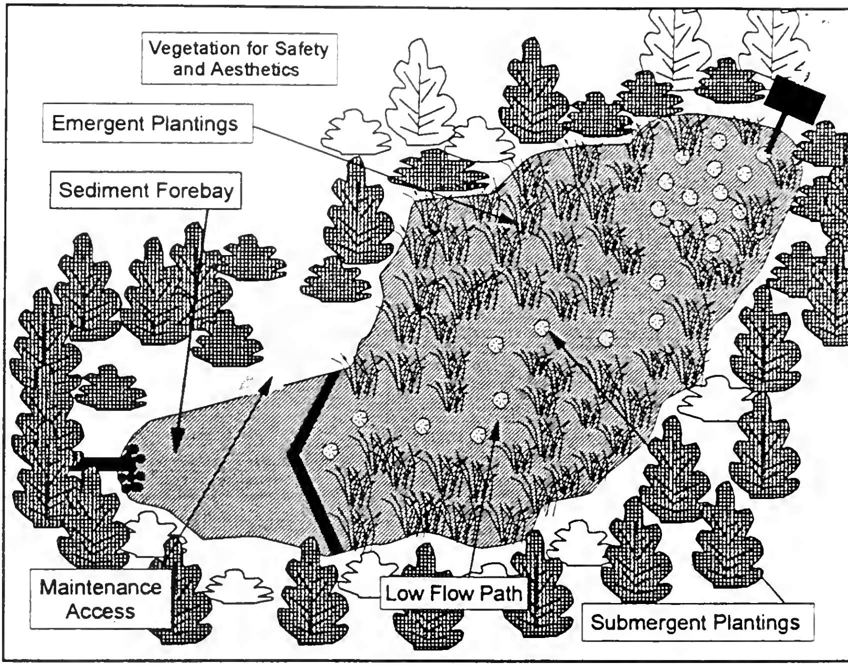


Figure 3.23 Extended Detention Wetland with Forebay

As a result of the lack of knowledge concerning constructed wetlands and their impacts, the recently revised Municipal Class Environmental Assessment (1993) indicated that wetlands would be subject to a Schedule C Class EA. This issue was subsequently clarified such that wetlands where the biological treatment processes have been engineered to achieve certain pollutant removal criteria would be subject to a Schedule C Class EA, while wetlands which were designed based primarily on hydrologic/hydraulic criteria would not be subject to the Schedule C requirements.

Although there are numerous concerns and unanswered questions associated with constructed wetlands, the limited monitoring of stormwater wetlands suggests that they are an effective SWMP for water quality enhancement.

Design Guidance

Guidance for stormwater wetlands can be classified into several groups :

- drainage area
- length to width ratio
- permanent pool and active storage depth variations
- planting strategy
- inlet and outlet configurations
- maintenance and performance enhancements

Drainage Area

Wetlands require a minimum drainage area to sustain the permanent pool. As a general rule wetlands should be implemented for drainage areas ≥ 5 hectares.

Detention Time

A detention time of 24 hours should be targeted in all instances, unless the potential for clogging the outlet is high.

In cases where the outlet is susceptible to clogging (ie. drainage areas < 8 ha - see minimum orifice size) the detention time can be reduced to a minimum of 12 hours. (Note : simulations that were performed in this study indicated that a 12 hour detention time produced similar settling to a 24 hour detention time)

The drawdown time in the pond can be estimated using Equation 3.2. Equation 3.2 is the classic falling head orifice equation which assumes a constant pond surface area. Although this is not strictly true, the calculation can be made using a linear regression relationship derived between the wetland surface area and wetland depth. A discussion of this equation is provided in Appendix E.

Minimum Orifice Size

The smallest diameter orifice accepted by most municipalities to ensure that clogging does not occur in a stormwater system is 75 mm. It is recommended that this minimum size be maintained for exposed outlet designs (ie. reverse sloped pipes). In instances where a perforated riser outlet is designed, the orifice is protected by the smaller perforations in the riser and a minimum orifice size of 50 mm is acceptable.

Permanent Pool and Extended Detention Storage Sizing

In areas with subwatershed planning, guidance should be provided with respect to both the permanent pool and active storage (extended detention) sizing. If a subwatershed plan has not been, and will not be, undertaken, the sizing of both storage components is detailed in Chapter 4. Sizing of these components was based on long term continuous modelling of pollutant removal performance (Appendix I).

Length : Width Ratio

The flow path through a wetland is important to the overall performance of this SWMP. In contrast to the wet pond, however, the flow path in a wetland is mostly dependent on the plantings and grading within the wetland due to the shallow depth of the permanent pool.

Although a length to width ratio of 3:1 is recommended in a stormwater wetlands it should be measured based on the flow path of low flows through the wetland rather than the overall dimension of the wetland itself. Low flow paths should be created through the wetland to ensure that short circuiting does not occur and that the flow path through the wetland is maximized during small events. Figure 3.24 illustrates the concept of providing low flow paths which maximize the flow path.

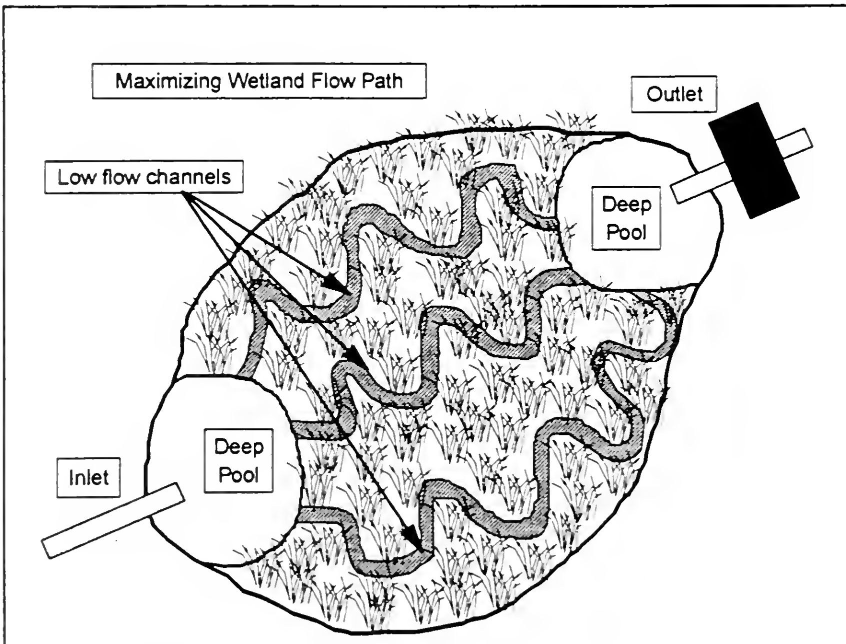


Figure 3.24 Maximizing Wetland Flow Path

All stormwater servicing should be conveyed to one inlet location, if possible, in order to facilitate maintenance.

Permanent Pool Depth

The average permanent pool depth in a wetland should range from 150 mm to 300 mm. Inlet and outlet areas should be deeper to minimize the re-suspension and discharge of settled pollutants from the facility. The maximum depth in the inlet and outlet areas should be restricted to 3 metres. It must be recognized that the deep inlet and outlet areas will be mainly open water since they will be too deep to sustain emergent vegetation. As such, the deep areas in wetlands should be limited to 25% (Livingston, 1989; Bradley and Cook, 1951) of the total surface area to ensure that the majority of the wetland sustains emergent vegetation.

Extended Detention Storage Depth

The extended detention storage depth should be limited to 1 metre. Stormwater wetlands consist of mostly planted areas. The depth restriction in the extended detention storage portion of the wetland is related to the planting strategy since some plant species cannot withstand water level fluctuations in excess of 1 metre. Although the depth of 1 metre is appropriate as a generic value for most wetland designs, the depth of the extended detention storage should be based on the planting strategy that is chosen for the wetland. As such, an aquatic biologist should dictate the desired extended detention design depth based on the proposed planting strategy.

Planting Strategy

A planting strategy is the most important aspect of a stormwater wetland. The planting strategy can be divided into five zones based on average water depth and soil moisture regime (frequency of wetting/inundation) :

1. deep water areas (submergent vegetation)
2. shallow water areas (emergent vegetation)
3. extended detention or shoreline fringe areas
4. floodfringe areas (if the facility is a combined quality/quantity SWMP)
5. upland areas

Deep Water Areas

In wetlands, the main deep water areas occur at inlets, outlets, and low flow paths. Plantings in deep water areas are restricted to submergent vegetation. This type of vegetation is difficult to establish using seeds or rhizomes. Pondweeds can be planted in water depths of 2 m - 3 m. Other submerged species (listed in Appendix F) can be planted in water depths between 1 m - 2 m. Shallow and deep water plantings will become established according to water level fluctuations and critical depths for light availability. It is expected that there will be some intergradation of species between the 1 m and 3 m depths.

Shallow Water Areas

Shallow water areas represent areas where the permanent pool is ≤ 0.5 metres deep. In wetlands this area represents the majority of the plantings. The selection of vegetation in these areas should be based on achieving several objectives :

- nutrient uptake
- filtration of stormwater
- maximizing the flow path through the wetland
- safety
- enhancing the aesthetics of the wetland

The plantings in this zone provide ancillary benefits including :

- prevention of re-suspension of bottom sediments
- reduction of flow velocities promoting sedimentation

This zone includes both submergent and emergent vegetation. Typical species are listed in Appendix F. Submerged species should be planted starting at a water depth of 0.5 m from the shoreline. The majority of submerged species should be planted in water depths between 0.3 m and 0.5 m. Emergents should be planted starting at a water depth of 0.3 m. Species such as sedges, reed grass, and arrowhead should be planted at the water's edge. The wetland side slopes constrain the amount of vegetation that can be established. A minimum slope is preferable to maximize the area available for plantings.

Shoreline Fringe Areas

Shoreline fringe areas represent the area which is subject to frequent wetting as a result of storm events. This area can be delineated by the land between the permanent pool and high water mark (extended detention storage) for erosion/water quality control. This land will be subject to higher soil moisture conditions as a result of water level fluctuations during storm events and the influence of the permanent pool itself during dry weather conditions.

The objectives in the design of the planting strategy for the shoreline fringe area are similar to those in the shallow marsh area. The growing conditions in the fringe area, however, are harsher than the marsh area due to the frequent wetting/drying cycle which the plants must endure. Consequently, this area requires close attention during the design phase as well as implementation stage to ensure that the proposed plants become established.

Plant material in the shoreline fringe areas includes hardy hydric grasses and shrubs. A list of common species is provided in Appendix F. The grasses should be sown using a mixture in the spring or fall (spring is preferable) from the water's edge of the

permanent pool to the limit of the extended detention. The shrubs should be planted in the extended detention zone such that only their lower branches will be inundated during the design storm. At least two shrub species should be planted to improve survival success.

Given the harsh growing conditions in this zone, it may take a considerable time to establish the vegetation, and replanting may be periodically required.

Floodfringe Areas

If the wetland is also used to control peak flow rates during infrequent storm events (2 year to 100 year), a zone of infrequent inundation will be created (floodfringe area). The influence of the permanent pool and frequent storm events is less pronounced for this area. The planting strategy in this zone includes a range of grass, herb, shrub, and tree species. A commercially available grass and herb seed mixture suitable for slope stabilization is recommended. Typical shrub and tree species are listed in Appendix F.

The grass and herb seed mixture should be sown in the spring or fall (spring is preferable). At least three species of shrubs and three species of trees should be planted in this zone. There should be a gradated change in planting near the upland zone to maximize the aesthetics in this area.

Upland Areas

Upland areas represent the landscaped areas provided as aesthetic amenities around the wetland. Although the plantings in this area are mainly a function of aesthetics, a primary objective with plantings in upland areas for stormwater wetlands is to provide an effective wind barrier for the wetland. The shallow permanent pool in a stormwater wetlands will be susceptible to wind turbulence, especially in the first 2 years of operation until the emergent vegetation becomes established.

Upland plantings should also be designed to restrict access to steep areas or inlet/outlet locations. A mixture of 5 plant species should be planted in a random pattern to prevent the establishment of monoculture areas. A list of typical species is presented in Appendix F. A large number of young plant stocks, tree whips and seedlings, should be implemented rather than a small number of large shrubs and caliper trees. Some caliper trees and mature plants should be used, however, to provide immediate wind screening, aesthetic, and safety objectives. Planting masses should be strategically located to minimize temperature increases in the wetland. The selection of plant materials should consider :

- the topography and surface drainage
- soil conditions
- adjacent plant communities

- the potential for on-site transplantation
- the availability of nursery stocks

A naturalized landscape approach should be used which requires no maintenance and is sustainable over the long term. The upland plantings should provide a minimum of a 3 m buffer strip from the maximum design water level mark. The massing of trees and shrubs should be augmented by designated regeneration areas to achieve long term growth.

Fencing

The use of permanent fencing is left to the discretion of the local municipality. Fencing prevents the integration of the wetland within a natural setting, however, and defeats the purpose of using natural system designs for water management. In areas where wetlands adjoin the valley corridor, fencing will impede wildlife passage. Therefore, it is recommended that natural solutions be investigated (ie. grading and planting strategies noted in next section) to build in safety features for the pond.

Temporary fencing may be required after the initial construction period until the vegetation is established. In these situations post and wire fencing is recommended. The openings in the fence should be ≥ 150 mm × 150 mm. The temporary fencing should be removed once the vegetation in the planting strategy is established, and the cost of removal included in the development agreement.

Signs around the wetland indicating it's purpose and function also help to inform the public of the potential for water level fluctuations during storm events.

Grading

By the nature of the limited allowable permanent pool and extended detention depths, grading in a wetland should be reasonably flat. The side slopes near the permanent pool should be 5:1 or flatter. Slopes in the extended detention portion of the wetland should not exceed 3:1.

Terraced grading (ie. a section of 7:1 grading, then 3:1 grading, then 7:1 grading) (Figure 3.12) is recommended to minimize the potential for the public to fall into the wetland.

Inlet Configuration

The stormwater conveyance system (sewers, grassed swales) should ideally have one discharge location into the wetland. This requires planning and ongoing interaction between land use planners and municipal engineers to ensure that it is economically efficient and feasible to drain the tributary area to one inlet location.

Exposed pilot channels (typically rock lined channels which convey stormwater from a pipe

outlet to the wetland) should be avoided since monitoring which has been performed indicates that these channels increase water temperature by 1°C for every 75 metres of pilot channel (Galli, 1990).

Inlets to wetlands without a forebay will be unsubmerged given the shallow depth of the permanent pool. Unsubmerged inlets are generally easier to design since they do not introduce hydraulic complications into the system. The invert of the inlet pipe is set at the maximum design water level in the wetland (assuming no flow splitter upstream, flow splitters are discussed in Section 3.5).

As a result of the raised inlet invert and wetland side slopes, the unsubmerged inlet will not discharge directly into water. It is important that the erosion potential between the inlet and the permanent pool be addressed in this design. The use of environmental stone/blocks (interlocking blocks with large openings to allow vegetative growth in between the blocks) in this area is recommended (Figure 3.21) in this regard since they will minimize the erosion potential.

Outlet Configuration

The outlet configuration options for a wetland are the similar to those for a wet pond (Section 3.4.3).

Reversed Sloped Pipe

A reverse sloped pipe (Figure 3.15) is appropriate for wetlands with outlet areas ≥ 1 m deep. The reverse sloped pipe is used as the outlet in the water quality/erosion portion of the wetland. The reverse sloped pipe should drain to an outlet chamber located in the wetland embankment. The outlet chamber can contain openings for flood control detention and overflow protection. It is recommended that a gate valve be attached to the reverse sloped pipe in the outlet chamber. This valve will allow the extended detention drawdown time to be modified to improve pollutant removal if the pond is found to be operating outside of the design criteria.

A low flow maintenance pipe should be provided to drain the wetland for maintenance purposes. The maintenance pipe should also drain to the outlet chamber. It is recommended that the maintenance pipe be sized to provide a 6 hour detention time (6 hours was chosen as a reasonable time period in which to drain the entire pond for maintenance recognizing that the release rate should not affect the downstream receiving waters), and that a gate valve be attached to the end of this pipe in the outlet chamber.

Perforated Riser

A perforated riser design is recommended for wetlands in which there is no deep outlet pool. Figure 3.16 illustrates such a design.

In cases where there is a lengthy outlet channel natural channel design techniques should be employed. Guidance on natural channel design techniques is provided in "Natural Channel Systems : An Approach to Management and Design - Development Draft" (Ministry of Natural Resources, 1994).

Maintenance and Performance Enhancements

There are several guidelines which will improve the performance of a wetland with respect to pollutant removal and facilitate maintenance activities :

- sediment forebay
- hard bottomed access for maintenance

Sediment Forebay

A sediment forebay facilitates maintenance and improves pollutant removal by trapping larger particles near the inlet of the wetland.

A forebay is especially important in a wetland design since maintenance will be restricted to this area (for the most part), minimizing the need to disturb the wetland vegetation. The forebay should be deep (≥ 1 metre) to minimize the potential for scour and re-suspension.

The forebay sizing depends on the inlet configuration, and several methods for sizing are provided in Section 3.4.3 (Wet Ponds). In all instances the forebay should not exceed one fifth of the wetland surface area.

Forebay Berm

The forebay should be separated from the rest of the wetland by an earthen berm. The berm should be set at the permanent pool elevation or extend into the extended detention portion of the wetland to act as a level spreader during storm events, and to minimize the disruption to the wetland during maintenance of the forebay. Flow calculations should be made under design conditions to ensure that the berm does not provide a flow restriction which would cause the whole forebay to overflow (not just into the wetland) because of the restriction of flow into the wetland resulting from a small berm width.

If a by-pass pipe is proposed to convey high flows around the wetland, a maintenance pipe should be installed in the forebay and connected to the bypass pipe, if grades permit, to drawdown the forebay for maintenance purposes.

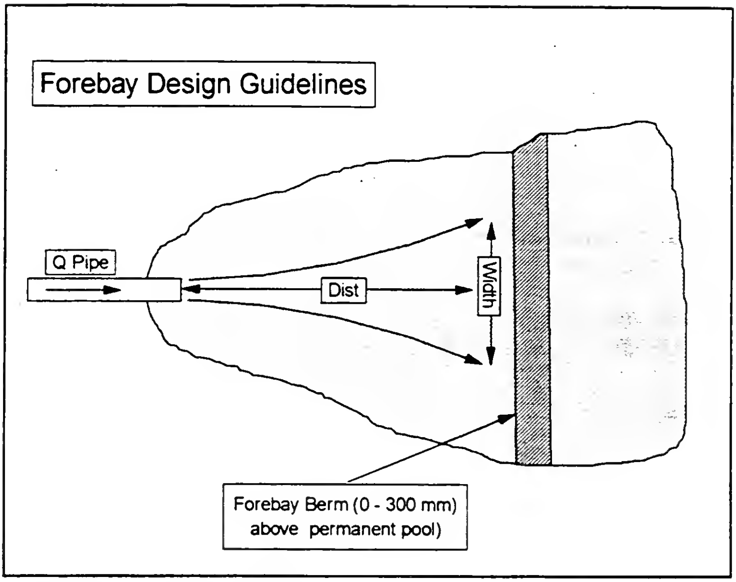


Figure 3.25 Wetland Forebay

The berm height should be set at, or within 0.30 metres of, the permanent pool elevation in the wetland. Suitable species of plants include fragrant waterlily, american bulrush, and softstem bulrush. The plants should be established on the top and sides of the berm at a maximum planting depth of 30 cm.

Hard Bottomed Surface

Experience in Ottawa with wet ponds indicated many operational and maintenance problems (Gietz, 1983). One of the problems was removing sediment which accumulated in the ponds. Although the ponds were drawn down to allow vehicular access for maintenance, natural pond bottoms proved to be too soft and hindered maintenance. In order to facilitate maintenance it is recommended that wetland forebays be lined with a hard-bottomed surface. The surface should have openings to allow the growth of plant material, but also be capable of withstanding the weight of small grading equipment which will be used to maintain the forebay. Several manufacturers currently distribute this type of product in Ontario. The hard bottomed surface should extend to the vegetated shoreline fringe in the forebay area. One or two access routes into the wetland should be constructed using the hard bottom surface.

A hard-bottomed surface can also be installed near the outlet of the wetland to provide an access route to the outlet pipes for maintenance. Although desirable, this treatment is less important than providing a hard-bottomed access route near the inlet to the forebay.

Technical Effectiveness

Wetlands which incorporate extended detention are encouraged for implementation. As with any SWMP, there are both advantages and drawbacks to the use of wetlands for stormwater management. They are the most land consumptive end-of-pipe SWM facility which can be chosen, and at the same time, are one of the most effective SWMPs for water quality enhancement.

Wetlands can and should be used for erosion control and water quantity control purposes. The restriction on extended detention storage depth (I metre), however will necessitate a large surface area for flood control purposes. In addition, flood control (quantity) is discouraged for wetlands that do not incorporate a sediment forebay.

3.4.6 Infiltration Trenches

Infiltration trenches in this manual refer to infiltration systems with a subsurface storage component that treat stormwater runoff from several lots/properties as opposed to soakaway pits which are primarily used for a single lot application.

In the past, infiltration trenches have been used to treat stormwater runoff from an entire site (including roads and parking lots). Infiltration trenches are comprised of a clear stone storage layer and a sand, or peat sand, filter layer. Pre-treatment SWMPs such as filter strips and oil/grit separators are generally implemented to minimize the potential for suspended sediments in stormwater to clog the trench.

Infiltration trenches can be implemented at the ground surface to intercept overland flows, or underground as part of the a storm sewer system. Infiltration trenches will provide marginal water quantity and erosion benefits but should not be implemented with these goals as their prime objective.

Design Guidance

Drainage Area

Infiltration trenches should be implemented for small drainage areas (< 2 ha). The use of trenches for larger drainage areas is inappropriate due to the technical infeasibility of infiltrating a large volume of water in a relatively small area of land. Groundwater mounding problems, compaction, and sealing of the native soii material have a higher potential of occurrence as the volume of water to be infiltrated increases.

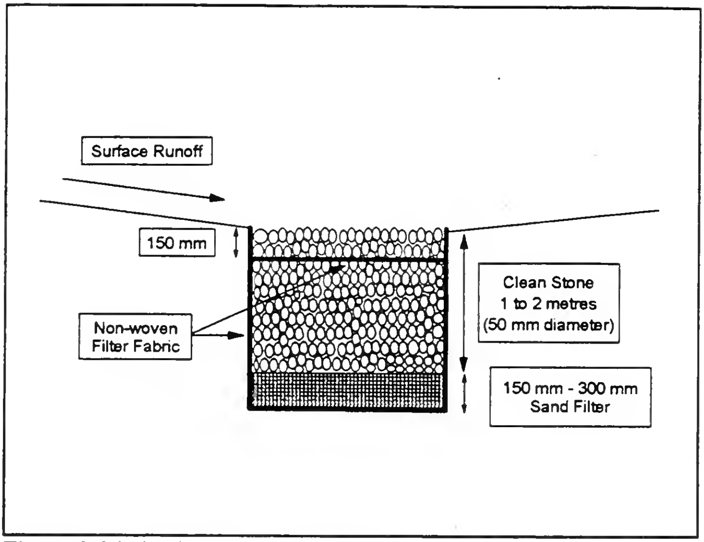


Figure 3.26 Surface Infiltration Trench

Land Use

Infiltration trenches should be implemented for residential land uses. Trenches are best implemented for compact housing (cluster housing, townhouses) in small parks/greenspace areas where several households can drain to a single trench.

Infiltration trenches are not recommended for industrial land uses since there is a high potential for groundwater contamination and/or dry weather spills. Similarly, infiltration trenches are not recommended for commercial parking lots since there is a high potential for dry weather spills and chlorides to enter the trench, and subsequently, the groundwater system. In site specific cases where infiltration trenches are deemed acceptable for these land uses, the design should be off-line (ie accept low flows only) and incorporate some form of upstream pre-treatment (a filter strip, oil/grit separator and/or sand filter).

Soils

Infiltration trenches are not recommended for areas in which the native soil has a percolation rate of less than 15 mm/h. Typical percolation rates and soil types are provided in Table 3.1. Table 3.1 should be used as a screening tool to determine if a site may be suitable for an infiltration trench. If a site is acceptable based on the screening process, individual site testing should be undertaken to determine whether the actual percolation rates for the soil are suitable for an infiltration trench.

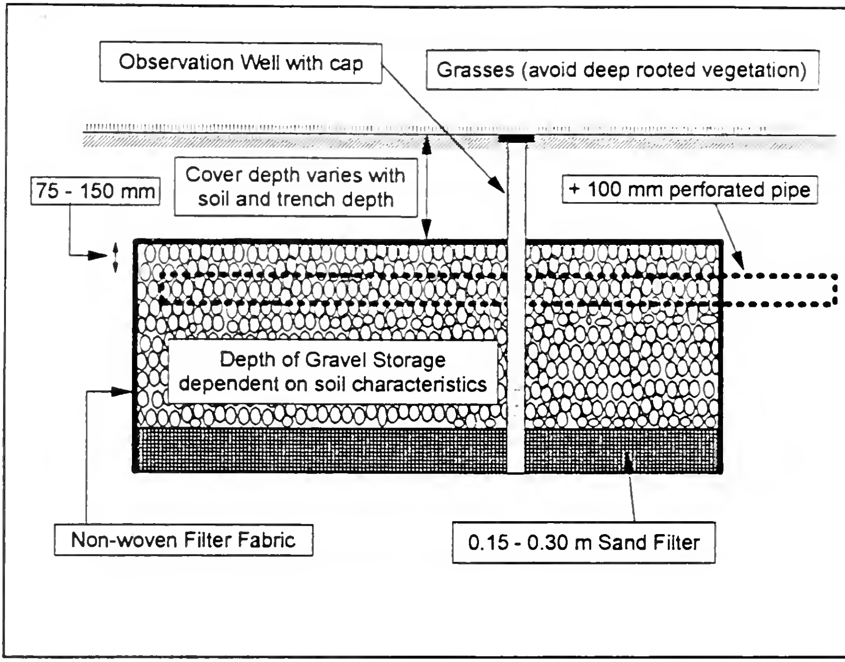


Figure 3.27 Subsurface Infiltration Trench

Water Table Depth

The seasonally high water table depth should be > 1 m below the bottom of the infiltration trench.

Bedrock Depth

The depth to bedrock should be > 1 m below the bottom of the infiltration trench.

Storage Media

The largest feature of an infiltration trench is storage to hold the stormwater until it can percolate into the surrounding native material. It is recommended that 50 mm diameter clear stone be used as the storage media in an infiltration trench. Non-woven filter fabric should be implemented at the interface of the trench and the surrounding native material to ensure that the native material does not clog the trench. If a subsurface trench is implemented the filter fabric should extend to cover the top of the trench.

If a surface trench is implemented there should be a layer of non-woven filter fabric 150 mm to 300 mm below the ground surface to prevent suspended solids from clogging the majority of the storage media. It should be recognized, however, that there may be a need to frequently replace this filter fabric layer depending on the volume of suspended solids transported to the trench.

It should be noted that while gravel is the most common form of storage in infiltration trenches, there are suppliers that manufacture precast infiltration storage media. These alternative storage media solutions are generally acceptable and should be reviewed and implemented on a case by case basis until there is adequate research/experience with their performance.

Filter Layer

A filter layer is constructed underneath the storage layer to provide tertiary treatment of the stormwater before it infiltrates the native soil. The most common filter media used in infiltration trenches is sand. The sand layer should be approximately 0.3 m thick (0.15 m - 0.30 m). The recommended gradation of sand to use is based on the effective size (10% of the particles are less than this size) and the coefficient of uniformity (C_u) (the larger the number the less uniform the material). Sands with an effective size of 0.25 mm with a $C_u \leq 3.5$, or an effective size of 2.5 mm with a $C_u \leq 1.5$, are recommended for filter material (Chapter 403, Florida Statutes).

A mixture of peat with the sand can be used to enhance the pollutant removal characteristics of the trench. Peat has a high affinity for metals, hydrocarbons, and nutrients (Galli, 1989). Fibric or hemic peat should be utilized to achieve the desired percolation rates. Sapric peat is discouraged since this type of peat has a slow percolation rate. If a peat-sand mixture is used for the filter media, the percolation rate of this media should equal, or exceed, the percolation rate of the surrounding native soil material.

Pre-Treatment

If a surface trench is chosen, there should be a 20 m wide (minimum) filter strip upstream of the trench to provide some pre-treatment of the stormwater (see Section 3.4.7) before it enters the trench.

If a subsurface trench is used, pre-treatment in the form of grassed swales, a filter strip, a sand filter, and/or a dry weather oil/grit interceptor should be implemented before the stormwater enters the trench.

Trench Configuration

The depth of the storage layer should be sized to ensure a 24 to 48 hour drawdown of the stored water based on the percolation rate determined in the field (24 hour is recommended). Equation 3.1 can be used to calculate the maximum allowable storage depth in the trench.

The length and width of the trench will be determined by the characteristics of the site in question (topography, size and shape). A maximum storage volume equal to the runoff from a 4 hour 15 mm storm should be provided in the trench storage media if the trench accepts runoff from several lots.

If a surface trench is designed, the dimensions of the trench will depend on the path of influent water. If stormwater is conveyed to the trench as uniform sheet flow, the length of the trench perpendicular to the flow direction should be maximized. If stormwater is conveyed as channel flow, the length of trench parallel to the direction of flow should be maximized.

In a subsurface trench, the water is conveyed into the trench via a pipe system. In this arrangement, it is recommended that the trench length (parallel to the incoming pipe) be maximized compared to the trench width. This will encourage the uniform distribution of water in the storage layer.

The appropriate bottom area of the trench can be calculated using Equation 3.8. This equation assumes that all of the infiltration occurs through the bottom of the trench.

$$A = \frac{1000V}{P\ n\ \Delta t}$$

Equation 3.8 Infiltration Trench Bottom Area

where A = bottom area of the trench (m^2)
V = runoff volume to be infiltrated (Table 4.1)
P = percolation rate of surrounding native soil (mm/h)
n = porosity of the storage media (0.4 for clear stone)
Δt = retention time (24 to 48 hours)

Distribution Pipes

In an underground trench, water is conveyed into the storage layer by a series of perforated pipes. A header pipe connects to the influent storm sewer and distributes the flow into lateral perorated pipes (≥ 100 mm diameter) which traverse the entire length of the trench. The lateral pipes should be spaced a maximum of 1.2 m apart. The perforated pipes should be located approximately 75 mm - 150 mm from the top of the storage layer.

Overflow/Bypass Pipe

A bypass pipe should be incorporated in the design of an underground infiltration trench to convey high flows around the trench. This will necessitate the construction of a flow splitter upstream of the trench (see Section 3.5). The bypass pipe serves several functions :

- used as the normal outlet until the site is stabilized (inlet pipe to trench blocked off)

- used as the normal outlet during trench maintenance
- used as the normal outlet during winter/spring conditions

Groundwater Mounding

Calculations to determine groundwater mounding may be necessary in cases where slope stability is a concern, and/or a high water table is encountered. A hydrogeologist should be consulted with respect to the potential for groundwater mounding in these areas. The results from groundwater mounding calculations should be regarded as an indication of the mounding potential rather than as an accurate representation of the actual mounding depth. The groundwater mounding calculations which are currently being used were designed specifically for septic tile fields, and have limited applicability for stormwater applications.

Proximity to Septic Fields

Groundwater mounding calculations may be required to ensure that infiltration trenches do not interfere with Class 4 and 6 sewage system leaching beds. A hydrogeologist should be consulted with respect to the necessity for mounding calculations and the requirements for a setback from the tile field. It is anticipated that calculations will be required in areas where the soils are marginally acceptable for infiltration. In areas with proposed EPA (Environmental Protection Act) Part VIII Program sewage systems, it will be necessary to consult with the approving Director of the Part VIII program. In areas where the percolation rate is high ($\geq$ 50 mm/h) groundwater mounding, and hence interference, will generally not be a problem.

Planting Strategy

The planting requirements for an infiltration trench are more aesthetic than functional. Plantings generally acceptable are grass/herb mixtures. Plantings with deep roots should be avoided since they can puncture the filter fabric at the top of the trench allowing native soil material to clog the gravel storage layer from above.

Construction

Infiltration trenches will only operate as designed if they are constructed properly. There are three main rules that must be followed during the construction of an infiltration trench :

1. Trenches should be constructed at the end of the development construction
2. Smearing of the native material at the interface with the trench must be avoided and/or corrected by raking / roto-tilling
3. Compaction of the trench during construction must be minimized

Maintenance and Performance Enhancements

To provide enhanced performance an infiltration trench design should provide :

- pre-treatment to reduce the sediment loadings to the trench
- off-line design / by-pass to restrict flows into the trench

Pre-treatment

Oil/grit separators are usually implemented as pre-treatment devices for infiltration trenches. Oil/grit separators are not highly effective sediment removal SWMPs. Effective pre-treatment SWMPs include wet ponds, dry ponds, wetlands, and sand filters. All of these end-of-pipe SWM facilities, except for sand filters, however, are not suitable for small drainage areas (<2 ha). Therefore, sand filters, vegetated filter strips, and/or oil/grit separators should be used to pre-treat stormwater which discharges to an infiltration trench, especially if road runoff is treated. Source controls should also be investigated (sanding/salting practices, public education with respect to street/driveway sediments) in areas where an infiltration trench is proposed.

By-Pass

A by-pass system should be implemented for all infiltration trenches. The bypass is useful from the standpoint that the inlet to the trench can be blocked during periods of sanding/salting, when local excavation works/maintenance are taking place, or when maintenance of the trench/pre-treatment system is required. By-passes and flow splitters are discussed in Section 3.5.

Technical Effectiveness

Centralized infiltration trenches have a poor historical record of success (Lindsey et al., 1992; Metropolitan Washington Council of Governments, 1992). This lack of success is attributable to many factors :

- poor site selection (industrial/commercial land use, high water table depth, poor soil type)
- poor design (lack of pre-treatment, clogging by surrounding native material)
- poor construction techniques (smearing, over-compaction, trench operation during construction period)
- large drainage area (high sediment loadings, groundwater mounding)

There are many reasons why an infiltration trench can fail. One of the main problems with centralized infiltration trenches is that water from a large area is expected to infiltrate into a relatively small area. This does not reflect the natural hydrologic cycle and generally leads to

problems (groundwater mounding, clogging, compaction). By contrast, lot level infiltration provides dispersed infiltration over the entire development area. Since smaller volumes of water are infiltrated at each location the problems of groundwater mounding, clogging, and compaction from the weight of water, are less likely to occur.

It is for these reasons that centralized infiltration trenches are only recommended for limited application based on land use and drainage area. Infiltration trenches are promoted for compact development layouts where a group of lots/dwellings can drain to an infiltration trench in a common greenspace area. Infiltration trenches are generally not recommended for industrial and commercial sites due to the potential for chemicals and chlorides to enter the groundwater aquifer.

Similarly, surface infiltration trenches which service roads/parking lots are generally discouraged because of the ineffectiveness of pre-treatment devices during the winter and spring, the heavy sediment loads during the spring, and the high chloride inputs during the winter and spring.

Infiltration trenches should be designed with a bypass system for large flows. Trenches are ineffective quantity control facilities and should not be used for this purpose.

Consideration should be given to the operation of infiltration trenches during the winter period. For example, the inlet to underground trenches which accept road runoff should be closed during the winter period to ensure that the winter sanding of roads will not clog the infiltration trench.

To summarize, infiltration trenches are best utilized in compact residential development situations where a group of homes can drain to one central area. It is recommended that infiltration be used more for the recharge of water (non-road/parking lot impervious areas (rooftops, walkways), and pervious areas) than for water quality enhancement. Infiltration trenches require adequate pre-treatment and are therefore ideal secondary SWM facilities (ie. wet/dry pond or wetland discharging to a small infiltration trench). Infiltration trenches should incorporate a by-pass and should not be used primarily for water quantity control.

3.4.7 Infiltration Basins

Infiltration basins are above-ground pond systems which are constructed in highly pervious soils. Water infiltrates the basin and either recharges the groundwater system or is collected by an underground perforated pipe network and discharged to a downstream outlet. Infiltration basins which have been constructed have not been successful. Failure rates for infiltration basins in the United States range from 60% - 100% (Schueler, 1992). Many of the infiltration basins constructed in southern Ontario (ie. Guelph) have permanently standing water in them. The high failure rate can be attributed to :

- poor site selection (industrial/commercial land use, high water table depth, poor soil type)

- poor design (depth of ponding)
- poor construction techniques (smearing, over-compaction, basin operation during the construction period)
- large drainage area (high sediment loadings)
- lack of maintenance

Design Guidance

Drainage Area

Infiltration basins should be implemented for small drainage areas (< 5 ha). Although infiltration basins were originally designed to accommodate larger drainage areas, the monitoring which has been undertaken to-date indicates that large scale infiltration is not feasible.

Land Use

Infiltration basins should be implemented for residential land uses. Infiltration basins are not recommended for industrial and commercial land uses since there is a high potential for groundwater contamination from chemical spills and maintenance (salting/sanding) activities. In site specific cases where infiltration basins are deemed acceptable for these land uses, the design must be located off-line and incorporate some form of upstream treatment (an upstream oil/grit separator or sand filter).

Soils

Infiltration basins are not recommended for areas in which the native soil has a percolation rate of less than 60 mm/h. Typical percolation rates and soil types are provided in Table 3.1. Table 3.1 should be used as a screening tool to determine if a site may be suitable for an infiltration basin. If a site is acceptable based on the screening process, individual site testing should be undertaken by a qualified soils specialist or hydrogeologist to determine whether the insitu soil percolation rates are suitable for an infiltration basin.

Water Table Depth

The seasonally high water table depth should be greater than 1 m below the bottom of the infiltration basin.

Bedrock Depth

The depth to bedrock should be > 1 m below the bottom of the infiltration basin.

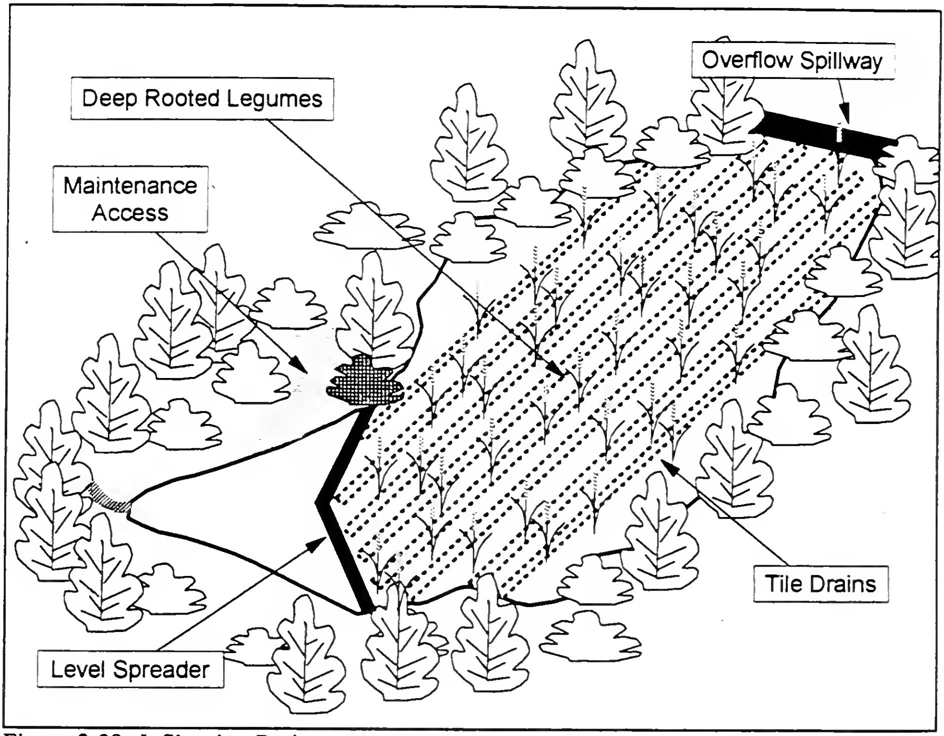


Figure 3.28 Infiltration Basin

Storage Configuration/Depth

In an infiltration basin, surface storage is used to retain water for infiltration. In recent monitoring studies (Galli, 1992) one of the causal factors of failure was noted to be the depth of water retained in the basin. The weight of the water is thought to compact the basin decreasing its infiltration potential.

The depth of storage should be limited to a maximum of 0.6 metres in order to minimize the compaction of the basin.

The length and width of the basin will be determined by the characteristics of the site in question (topography, size and shape). A desirable length to width ratio for an infiltration basin is 3:1 or greater. The appropriate minimum bottom area of the basin can be calculated based on Equation 3.8 with the porosity (n) set equal to 1.

Overflow / Bypass

A by-pass flow path/pipe should be incorporated in the design of an infiltration basin to convey high flows around the basin. This will necessitate the construction of a flow splitter upstream of the basin (see Section 3.5). The by-pass serves several functions :

- used as the normal outlet until the site is stabilized (inlet to the basin is blocked off)
- used as the normal outlet during basin maintenance
- used as the normal outlet during winter/spring conditions

Groundwater Mounding

Calculations to determine groundwater mounding may be necessary in cases where slope stability is a concern, and/or a high water table is encountered. A hydrogeologist should be consulted with respect to the potential for groundwater mounding in these areas. The results from groundwater mounding calculations should be regarded as an indication of the mounding potential rather than as an accurate representation of the actual mounding depth. The groundwater mounding calculations which are currently being used were designed specifically for septic tile fields, and have limited applicability for stormwater applications.

Proximity to Septic Fields

Groundwater mounding calculations may be required to ensure that infiltration basins do not interfere with Class 4 and 6 sewage system leaching beds. A hydrogeologist should be consulted with respect to the necessity for mounding calculations and the requirements for a setback from the tile field. It is anticipated that calculations will be required in areas where the soils are marginally acceptable for infiltration. In areas with proposed Environmental Protection Act Part VIII Program sewage systems, it will be necessary to consult with the approving Director of the Part VIII program. Given the high percolation rates (≥ 60 mm/h) recommended for the implementation of infiltration basins, groundwater mounding generally should not be a problem.

Planting Strategy

The planting strategy in an infiltration basin should be able to withstand periods of ponding and maintain or enhance the pore space in the underlying soils. There is numerous literature to suggest that deep rooted legumes increase root porosity and enhance infiltration compared to other ground covers (ie. rotation of oat and corn crops with alfalfa to maintain porosity) (Bryant et al. 1986; Minnesota Pollution Control Agency, 1989). As such, the planting strategy should include grasses and deep rooted legumes.

Fencing

The use of permanent fencing is left to the discretion of the local municipality. Fencing prevents

the integration of the basin within a natural setting, however, and defeats the purpose of using natural system designs for water management. Therefore, it is recommended that natural solutions (ie. vegetation plantings, grading) be investigated to build in safety features for the basin.

Temporary fencing may be required after the initial construction period until the vegetation is established. In these situations post and wire fencing is recommended. The openings in the fence should be ≥ 150 mm × 150 mm. The temporary fencing should be removed once the vegetation in the planting strategy is established, and the cost of removal included in the development agreement.

Signs around the basin indicating the it's purpose and function also help to inform the public of the potential for water level fluctuations during storm events.

Construction

Infiltration basins will only operate as designed if they are constructed properly. There are three main rules that must be followed during the construction of an infiltration basin :

1. Basins should be constructed at the end of the development construction
2. Smearing of the native material at the interface with the basin floor must be avoided and/or corrected by raking or roto-tilling
3. Compaction of the basin during construction must be minimized

Maintenance and Performance Enhancements

There are two design requirements to enhance the performance of an infiltration basin :

- pre-treatment to reduce the sediment loadings
- a by-pass to restrict flows into the basin

Pre-Treatment

Effective pre-treatment SWMPs include wet ponds, dry ponds, wetlands, and sand filters. Wet facilities (wet ponds, wetlands) will require compaction and/or liners to ensure that a permanent pool can be maintained. For small drainage areas, sand filters or oil/grit separators should be used to pre-treat stormwater which discharges to an infiltration basin, especially if road runoff is treated. Source controls should also be investigated (sanding/salting practices. public education with respect to street/driveway sediments) in areas where an infiltration basin is proposed.

In areas where the soils are marginally acceptable, perforated pipes can be implemented to augment the drainage. Perforated pipes (100 mm diameter) which traverse the entire length of

the basin should be spaced a maximum of 1.2 m apart at a depth of approximately 300 mm - 600 mm below the ground surface. The perforated pipes should drain to a header pipe which discharges to an outlet pipe/structure. The outlet pipe/structure would discharge the collected water to the receiving water or outlet channel.

By-pass

A by-pass system should be implemented for all infiltration basins. The bypass could be utilized during periods of sanding/salting, when local excavation works/maintenance are taking place, or when maintenance of the basin/pre-treatment system is required. By-passes and flow splitters are discussed in Section 3.5.

Technical Effectiveness

Infiltration basins have an extremely poor historical record of success (Lindsey et al. 1992; Washington Council of Governments, 1992). One of the main problems with centralized infiltration basins, like infiltration trenches, is that water from a large area is expected to infiltrate into a relatively small area. This does not reflect the natural hydrologic cycle and generally leads to problems (groundwater mounding, clogging, compaction). By contrast, lot level infiltration provides dispersed infiltration over the entire development area. Since smaller volumes of water are infiltrated at each location, the problems of groundwater mounding, clogging, and compaction from the weight of water, are less likely to occur.

It is for these reasons that infiltration basins are generally discouraged.

In site specific instances where infiltration basins are allowed, they should be designed with a bypass system for large flows. As such infiltration basins are ineffective for water quantity control.

Consideration should be given to the operation of infiltration basins during the winter period. Winter sanding of roads will clog an infiltration basin, and winter salting will increase the potential for the contamination of groundwater with chlorides.

Infiltration basins require adequate pre-treatment and are therefore better secondary facilities (ie. wet/dry pond or wetland discharging to a small infiltration basin).

3.4.8 Filter Strips

Filter strips are engineered stormwater conveyance systems which treat small drainage areas. Generally, a filter strip consists of a level spreader and numerous vegetative plantings. The level spreader ensures uniform flow over the vegetation. The vegetative plantings filter out pollutants, and promote infiltration of the stormwater.

There are two generalized types of filter strips : grass filter strips, and forested filter strips. There is a need for further research comparing the efficiency of these two types of filter strips for water quality enhancement, since the research to-date has focused on the individual assessment of these filter strip types.

Filter strips are best implemented adjacent to a buffer strip, watercourse or drainage swale since the discharge from a filter strip will be in the form of sheet flow, and therefore difficult to convey downstream in a normal stormwater conveyance system (swale or pipe).

Design Guidance

Drainage Area

Filter strips are feasible for small drainage areas (<2 ha).

Level Spreader

The level spreader consists of a raised weir constructed perpendicular to the direction of flow. Water is conveyed over the spreader as sheet flow to maximize the contact area with the vegetation. Although the spreader can be engineered using concrete, more natural spreader designs/materials are recommended to maintain a natural appearance and functionality.

Figure 3.29 illustrates a typical level spreader design. A small berm is used as the level spreader. The spreader creates a damming effect, preventing stormwater from entering the vegetation until the water level exceeds the height of the spreader. A perforated pipe (100 mm diameter) is implemented in the spreader berm to ensure that any water which is trapped behind the berm after a storm can be drained. The perforated pipe should be wrapped in a filter sock to ensure that the surrounding native material does not infiltrate the pipe.

The length of the level spreader should be chosen based on the site specifics of the area where it will be implemented (topography, outlet location, drainage area configuration). It should be recognized, however, that a shorter level spreader will result in a trade-off of greater upstream storage to maintain the desired flow depth over the vegetation (see storage and flow depth guidelines). It is recommended that the level spreader length, and hence filter strip length, be as long as possible to maximize the enhancements provided by the filter strip.

Flow Depth

The level spreader and filter strip should be designed such that the peak flow from a 4 hour Chicago 10 mm storm results in a flow depth of 50 - 100 mm through the vegetation. The flow depth over the level spreader can be calculated using a standard broad crested weir equation (Equation 3.9).

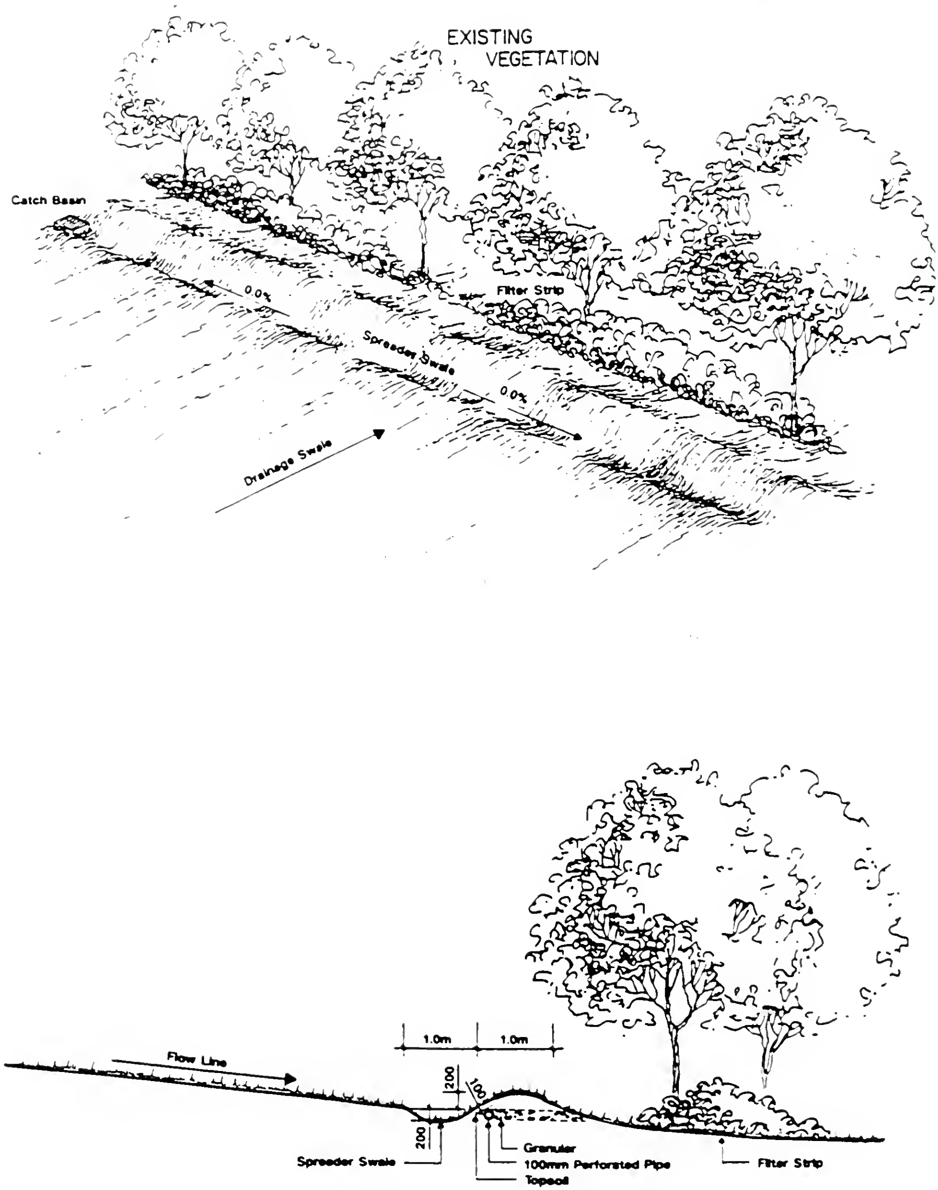


Figure 3.29 Typical Filter Strip

$$Q = 1.67\ L\ H^{1.5}$$ Equation 3.9 Weir Flow

Storage

Storage will be required behind the level spreader depending on the level of control desired, and the length of the level spreader itself. It is recommended that the amount of storage required should be based on the excess runoff from a 4 hour Chicago distribution of a 10 mm accounting for the flow over the weir. The 10 mm storm was chosen recognizing that 70% of all daily precipitation depths are less than or equal to this amount (Appendix C).

Vegetation

The vegetated strip should be 10 - 20 m wide in the direction of flow to provide sufficient stormwater quality enhancement (Osborne et al., 1993; Metropolitan Washington Council of Governments, 1992; Minnesota Pollution Control Agency, 1989). The slope of the vegetated strip should dictate the actual width. Species such as red fescue, tall fescue, and redtop can be introduced, in addition to the natural surrounding vegetation, to filter out stormwater pollutants. Species native to the area should be used, where commercially available, in the planting strategy.

Slope

Filter strips should be located in flat areas (< 10%) to promote sheet flow and maximize the filtration potential. The ideal slope in a filter strip is < 5 % (1% - 5%). There is a direct relationship between filter slope and filter length. Shorter filter strip widths (10 m - 15 m) are appropriate for flat slopes whereas longer filter strips (15 m - 20 m) are required in areas with a higher slope (5% - 10 %)

Technical Effectiveness

Vegetated filter strips have limited effectiveness for water quality control due to the difficulty of maintaining sheet flow through the vegetation. They are best implemented as one of a series of SWMPs in a stormwater management plan.

3.4.9 Sand Filters

Sand filters are a relatively new end-of-pipe SWM facility for Ontario. They have been used extensively in Dallas, Texas for the past 10 years with good success (Metropolitan Washington Council of Governments, 1992). Sand filters can either be implemented above ground, or below ground as part of the storm sewer system infrastructure and are generally intended for small drainage areas (≤ 5 ha).

Sand filters, as with all SWMPs, have both positive and negative attributes. They are effective

in removing pollutants, resistant to clogging, and are generally easier and less expensive to construct/retrofit than infiltration trenches. Sand filters do have some drawbacks, however. Unlike most infiltration system alternatives, sand filters do not recharge the groundwater system. Surface filters become unsightly as they trap pollutants, and can cause odour problems (Metropolitan Washington Council of Governments, 1992). In order to prevent these problems, sand filters require frequent maintenance, and hence, are more expensive to operate than other end-of-pipe SWM facilities.

Design Guidance

Drainage Area

Sand filters should be implemented for drainage areas ≤ 5 ha.

Location

Sand filters can be located either as surface facilities or as part of the storm sewer system. Stormwater can be conveyed to surface sand filters via overland flow or by a storm sewer. Sand filters can be implemented in the storm sewer system if there is enough topographical relief.

Sand filters are most commonly constructed with impermeable liners to ensure that native material does not enter the filter and clog the pore spaces, and to prevent the filtered water from infiltrating into the surrounding native soil material (ie. prevent groundwater contamination). Some surface sand filters include a grass cover to provide a more natural appearance. Figure 3.30 illustrates a typical cross-sectional view of a sand filter.

Storage Depth

It is recommended that the storage depth above a sand filter be limited to a maximum of 1 metre to reduce the potential for compaction of the sand layer.

Sand Filter Depth

The recommended sand filter layer depth is 0.5 m.

Sand Filter Discharge

Water which percolates through the sand filter is collected by pervious pipes and conveyed to the outlet. The pervious pipes should be 100 mm diameter drainage tile laid at the bottom of a 150 mm - 300 mm layer of 50 mm diameter gravel. The drainage tile should installed with a maximum spacing distance of 1.2 m. A layer of non-woven geotextile filter fabric should be installed between the sand layer and the gravel layer to minimize the potential for fines to clog the pore spaces in the gravel. The perforated pipes can be wrapped in non-woven filter fabric

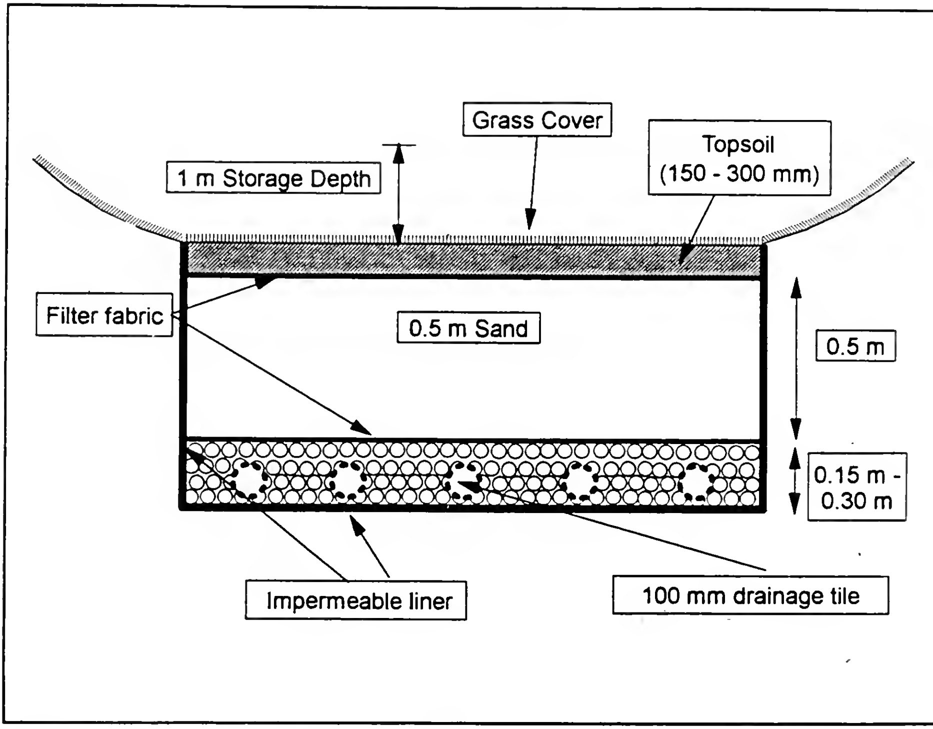


Figure 3.30 Sand Filter Cross Section Profile

(filter sock) for additional protection to prevent sand from entering the perforated pipe.

Overflow Discharge

All sand filters should include an overflow or by-pass conveyance system. A by-pass system which routes flows around the sand filter is preferable to an overflow at the sand filter itself since there is no permanent pool in a sand filter to mitigate influent velocities. The influent pipe to the sand filter should be designed to accommodate the peak flow from a 4 hour Chicago distribution of a 15 mm storm.

Maintenance and Performance Enhancements

One enhancement that can be incorporated into a sand filter design is the conveyance of the effluent discharge to a pervious pipe system or infiltration trench. If the sand filter discharges to a pervious pipe, a header pipe should collect the sand filter discharge and convey it to the

pervious pipe system. If the sand filter discharges to an infiltration trench, the individual collection pipes can be extended into the trench itself without the need for a collection header pipe.

A second enhancement is to include a layer of peat in the sand filter design. Peat has a high affinity for metals, nutrients, and hydrocarbons, providing a greater level of water quality enhancement.

The layer of peat is generally placed on top of the sand layer. The peat layer discharges to a peat-sand layer which provides a graduated increase in percolation to the sand layer. Figure 3.31 illustrates this type of design.

Fibric peat is recommended for implementation since this peat has a high infiltration rate. Hemic peat can be used but sapric peat is generally discouraged.

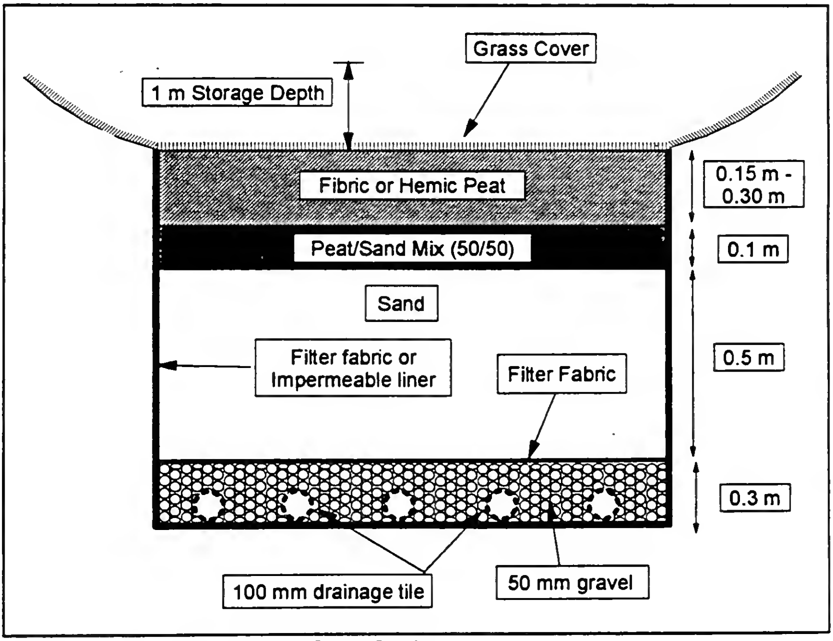


Figure 3.31 Peat Sand Filter Cross-Section

Technical Effectiveness

Sand filters have not been implemented on a widespread scale in Ontario. Little is known on their effectiveness during winter and freshet conditions.

Limited monitoring of a facility in Ottawa (Uplands Monitoring Report, 1992) indicates that sand filters are a promising stormwater management practice.

Sand filters should not be subjected to large influent flows. and therefore are not suitable for flood control purposes.

3.4.10 Oil/Grit Separators

A considerable amount of information has been collected with respect to oil/grit separators over the last several years. The increase in information can be attributed to the numerous installations of oil/grit separators, which in turn, can be attributed to the lack of SWM alternatives for drainage areas less than 5 ha.

Many development proposals are for small land areas (< 5 ha). In these situations the contributing drainage area is too small to implement wet ponds or wetlands. Recognizing that stormwater lot level controls are generally insufficient to control the currently accepted water quality requirements, some form of end-of-pipe SWM facility is usually necessary. End of pipe infiltration techniques (basins, trenches) are susceptible to clogging, and vegetative practices are only applicable for small drainage areas (< 2 ha) with little topographic relief. Therefore, only several SWMPs are feasible (sand filter, oil/grit separators) in many small development situations.

All oil/grit separators operate based on the principles of sedimentation for the grit, and phase separation for the oil. There is minimal attenuation of flow in oil/grit separators since they are not designed with extended detention storage. There are two general designs for oil/grit separators :

- 3 chamber separator
- manhole separator

Three Chamber Separator

The 3 chamber oil interceptor has been used in Maryland (USA) since 1987. The standard design was located on-line in the storm sewer and was subject to both low and high flow discharges. In most designs, the 3 chamber separator incorporates a permanent pool which is used for the capture of trash, dilution of pollutants during a storm, and the settling of material caught in the permanent pool during quiescent conditions after a

storm. A sizing rule of 30 m^3 per impervious hectare has been used in Ontario and Maryland for the permanent pool in this type of oil/grit separator. Figure 3.32 illustrates the typical three chamber design.

A recent monitoring study (Galli, 1990) found that the lack of extended detention, coupled with the on-line design resulted in poor performance. This study concluded that there was no significant sediment accumulation in the oil/grit separators over time, indicating that a continuous process of re-suspension/settling was occurring. The study also concluded that the majority of oil was either emulsified or bound to the fine sediments and could not be removed by the separator.

One of the major complaints against the 3 chamber separators is that they are cost ineffective given their large size and difficult/costly maintenance requirements. These separators require manual cleaning due to the large sizing rules applied in their design. As such, the maintenance of these structures is costly and may pose a safety hazard.

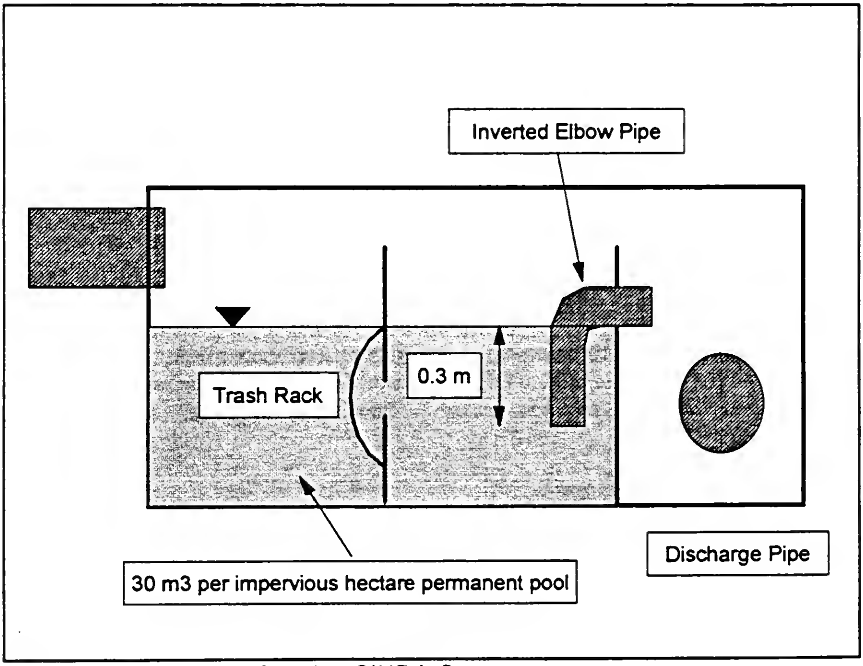


Figure 3.32 Standard 3 Chamber Oil/Grit Separator

The modelling of the end-of-pipe SWM facilities which was undertaken in this study, as

well as the monitoring information for oil/grit separators can be synthesized into two conclusions :

1. on-line oil/grit separators which do not incorporate a by-pass are ineffective
2. large oil/grit separators are cost ineffective stormwater quality controls and difficult to maintain

Design Guidance

Drainage Area

Three chamber oil/grit separators should be implemented for drainage areas ≤ 2 ha.

Land Use

Three chamber oil/grit separators should be implemented for commercial and industrial areas. Oil/grit separators are beneficial for large parking areas or transit stations (buses, passenger pick-up/drop off areas, service stations, etc.). As a general rule three chamber oil/grit separators should not be used as the primary water quality control for new residential developments since they are cost-ineffective in these areas.

Location

The performance of the three chamber interceptor can be greatly improved if an off-line design is implemented. An off-line design could incorporate either an upstream flow splitter or an orifice control/overflow system at the separator itself.

An upstream flow splitter can be designed using a parallel pipe system shown in Figure 3.33.

Separator Discharge Capacity

The pipe to the separator should be sized to accommodate low flows only. Low flows can be defined as flows smaller than the peak runoff from a 4 hour Chicago distribution of a 10 mm storm (An analysis of 20 years of the daily precipitation record from the Atmospheric Environment Service rainfall gauge at Yonge and Bloor Street indicated that a daily capture of 10 mm of precipitation would account for approximately 70% of the annual precipitation, see Appendix C). High flows would be conveyed by the flow splitter (discussed in Section 3.5) to the by-pass pipe. The by-pass pipe should be sized to convey the full pipe design storm (ie. 2, 5, or 10 year flows).

In situations where an offline three chamber oil/grit separator is proposed, the walls separating the three chambers can be extended to the top of the chamber to prevent

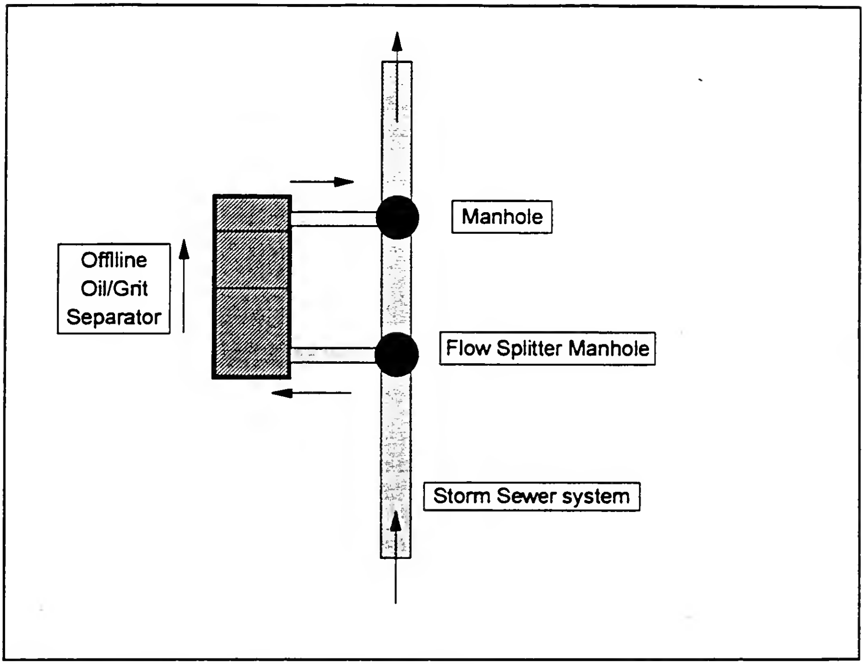


Figure 3.33 Offline Oil/Grit Separator Design

overflows from one chamber to the next. Upstream surcharging will not be a problem in this instance since the bypass pipe will be operating once the first chamber is full or the inlet pipe capacity is exceeded.

Length:Width Ratio

The length:width ratio of the flow path in a three chamber oil/grit separator should be 3:1 or greater.

Grit Chamber

The first chamber is designed to trap large grit particles. This chamber should be the largest chamber of the three. Fifty to seventy percent of the wet storage should be provided in the first chamber. A baffle should be constructed near the inlet of the chamber to reduce the flow velocity in the chamber, and hence minimize turbulence/re-suspension. An example of a baffle is shown in Figure 3.34.

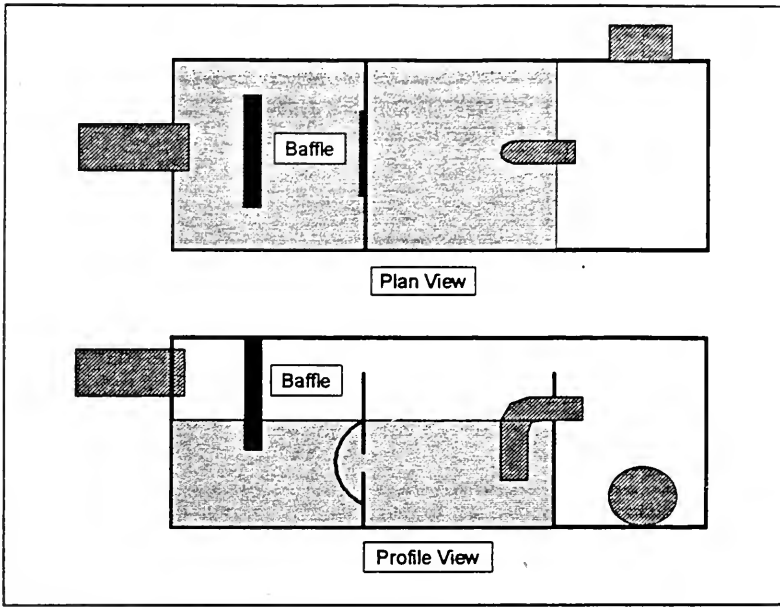


Figure 3.34 Oil/Grit Separator Inlet Baffle

The grit chamber is separated from the second/oil chamber by a trash rack located in the chamber wall. The trash rack consists of 25 mm diameter holes drilled in a metal plate (preferably aluminum). The area of the holes should be equal to 5% of the total chamber wall area. The flow area provided by the holes should be compared to the peak design inflow rate to ensure adequate capacity. Additional holes can be added if capacity is required as long as the total flow area is ≤ 8% of the wall area. This range of wall area openings (5% - 8%) will ensure uniform flow conveyance to the second chamber while minimizing turbulence.

The trash rack is bolted to the chamber wall. The wall area behind the trash rack should be cut away to allow water to freely enter the second chamber without increasing the flow velocities. The trash rack is best installed in sections which can be removed from the interceptor, if necessary, during maintenance.

<u>Oil Chamber</u>

The second chamber is provided primarily for oil removal, but also provides secondary grit/sediment removal. Water is conveyed from this chamber to the outlet by an inverted elbow pipe (Figure 3.32). Recent monitoring in Maryland (Galli, 1992) indicates that the elbow should extend approximately 300 mm into the permanent pool to minimize the

potential for siphoning of small sediments.

The inverted elbow should be oversized to reduce the velocity, and hence, potential for siphoning/discharge of sediment from the chamber. A minimum pipe size of 450 mm diameter should be utilized for the inverted elbow.

Discharge Chamber

The third chamber in the oil/grit separator is used simply as a discharge chamber to the receiving waters. This chamber is not required if a parallel pipe by-pass system is implemented. If the by-pass/overflow system is designed in the interceptor itself, this chamber will accommodate both normal interceptor flows and overflows/bypasses and discharge them to the receiving storm sewer. The outlet elevation from this chamber should ensure that there are no backwater effects on the other two chambers. If possible, the outlet should be raised from the chamber floor to provide a sump at the outlet.

Manhole Separator

Manhole oil/grit separators are pre-cast structures which take the place of a manhole. Figure 3.35 illustrates the design principles of these separators. Low flows enter a lower chamber where sedimentation and oil separation can occur. High flows bypass the lower chamber and flow through an upper chamber directly to the outlet pipe. The lower chamber is cylindrical with a circular bottom. An access chamber is provided which extends from the ground surface to the lower chamber directly. Cleanout of the lower chamber is performed using a vacuum truck. A typical manhole separator is shown in Figure 3.35.

The manhole shown in Figure 3.35 can be bought as a pre-cast unit in varying sizes and is manufactured in Ontario. Other interceptors, that are typically considered for combined sewer overflow (CSO) control can also be used. For example, vortex separators are available which discharge the retained solids directly to the sanitary sewer. This type of system would be appropriate in a highly urbanized area in the downstream portion of a watershed.

Design Guidance

Drainage Area

Manhole oil/grit separators should be implemented for drainage areas ≤ 1 ha.

Sizing

Manhole separators should sized based on providing 15 m^3 of permanent pool (wet)

storage for each impervious hectare of drainage area.

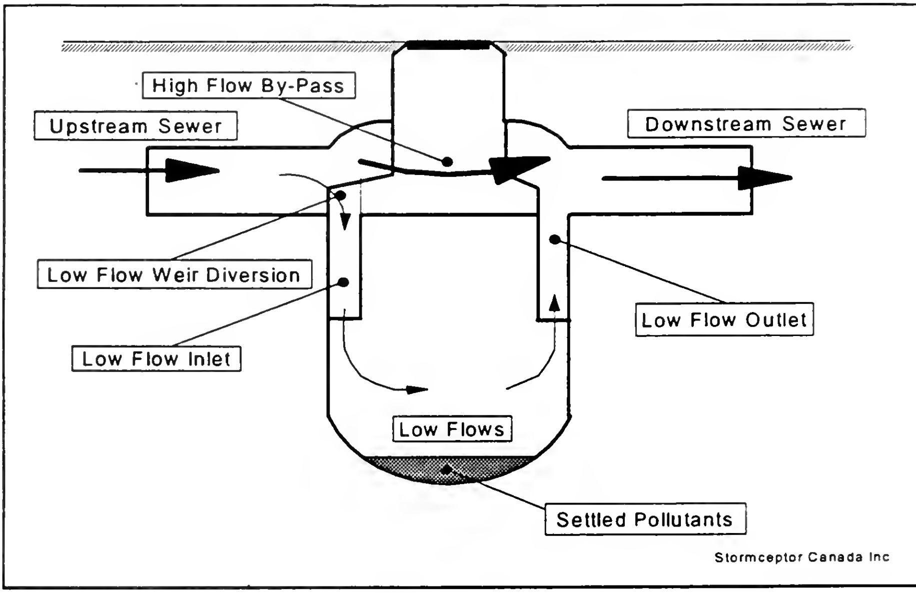


Figure 3.35 Manhole Oil/Grit Separator

Land Use

Manhole oil/grit separators should be implemented for commercial and industrial areas. Oil/grit separators are beneficial for large parking areas or transit stations (buses, passenger pick-up/drop off areas, service stations, etc.). As a general rule manhole oil/grit separators should not be used as the primary water quality control for new residential developments since they are cost-ineffective in these areas.

Location

Manhole separators are located on-line in the storm sewer system. They should be located downstream of any parking lot flow restrictors to prevent surcharging of the storage chambers during quantity events and to minimize peak flow rates through the units at all times.

Manhole separators are most effective for small drainage areas. If possible, they should be located on collector sewer lines before the collector joins the main sewer line (unless

a flow restriction is implemented downstream).

Technical Effectiveness

Monitoring of the standard three chamber oil/grit separators indicates that they are ineffective as stand-alone water quality enhancement measures. Enhanced design measures (by-pass, baffles) will improve the performance of these SWMPs. The greatest concern with these separators is that they are large underground structures, resulting in costly and difficult maintenance. The monitoring of on-line three chamber separators indicates that re-suspension is a problem since they do not accumulate pollutants over time. As such, the use of the three chamber interceptors is generally discouraged. In some site specific instances they may be acceptable if they are located off-line, and there is a binding agreement that maintenance will be performed.

Oil/grit separators are best utilized where there is the potential for dry weather spills/discharges. Smaller interceptors, such as the manhole type, are designed for low flow or spill situations. Accordingly, these interceptors are recommended for commercial parking lots and industrial land uses instead of the three chamber design.

Manhole separators are relatively new and are currently undergoing testing. As such, they are primarily recommended for low flows and spill control. Further research of these separators will indicate whether they provide greater benefits in terms of stormwater quality control.

Manhole separators can complement water quantity controls in situations where above-ground parking lot storage is designed. A manhole separator can be implemented at the manhole downstream of a flow restriction (orifice plate, small pipe) which causes surface ponding to occur. The manhole separator will benefit from the flow control provided by the flow restriction resulting in greater pollutant removal performance.

As a general rule, oil/grit separators are <u>not recommended</u> for low density residential land uses since it is anticipated that the greatest benefits resulting from their implementation will be with respect to spills control.

It is likely that an overall sediment removal performance of 20% to 50% can be expected depending on the size of separator implemented and the contributing drainage area. In order to provide adequate stormwater quality control, oil/grit separators are best implemented in a treatment train system where there are numerous water quality enhancements in series. Other water quality enhancements that will complement oil/grit separators include soakaway pits, vegetated filter strips, buffer strips, and enhanced grass swales.

These end-of-pipe SWM facilities are valuable for small, highly impervious areas that are susceptible to spills. Although the recent monitoring of first generation oil/grit separators indicates poor performance, second generation oil/grit separators will continue to be utilized in

many industrial and commercial development situations since there are relatively few stormwater management practices for small areas (<5 ha).

3.4.11 Porous Pavement

Porous pavement has been used in Canada, the United States, and Europe. Regular pavement is manufactured using a gradation of aggregate. Porous pavement is pavement which does not include fine aggregate material. The most common form of porous pavement today is primarily used for public safety purposes rather than stormwater management. Transportation agencies use porous pavement over a grooved impermeable subgrade surface. Water infiltrates the pavement but is channelled to the edges of the road by the impermeable subgrade. This design reduces hydro-planing but does little for stormwater management.

Early porous pavement designs incorporated porous pavement over gravel storage. Problems with compaction, clogging, lack of maintenance, and poor subgrade drainage have led to this practice being discouraged.

Given the climate in Ontario (long winters, salting and sanding practices) the use of porous pavement is generally discouraged. The historically high failure rates, potential for groundwater contamination by chlorides and spills, and high potential for clogging given Ontario sanding practices, all suggest that this SWMP not be actively implemented at this time.

3.4.12 Stream and Valley Corridor Buffer Strips

Buffer strips are simply natural areas between development and the receiving waters. There are two broad resource management objectives associated with buffer strips :

- the protection of the stream and valley corridor system to ensure their continued ecological form and functions
- the protection of vegetated riparian buffer areas within the valley system to minimize the impact of development on the stream itself (filter pollutants, provide shade and bank stability, reduce the velocity of overland flow)

Although both types of buffers provide only limited benefits in terms of stormwater management, they are an integral part of overall environmental management for sustainable development. The protection of stream and valley corridors provides significant benefits in terms of sustaining wildlife migration corridors, terrestrial and aquatic species food sources, terrestrial habitat, and linkages between natural areas.

Given the larger scale natural system benefits provided by stream and valley corridors, the required width of this type of buffer is best defined at the subwatershed plan level. In the absence of such a plan, Table 3.4 provides guidance with respect to an appropriate stream and

valley corridor buffer strip. Table 3.4 also indicates appropriate riparian buffer widths for the protection of the stream system itself. Table 3.4 was derived from the Metropolitan Toronto and Region Conservation Authority's Valley and Stream Corridor Management Program, the MOEE/MNR Interim Stormwater Quality Guidelines (1990), and the Ministry of Natural Resources document "Guidelines on the use of 'Vegetated Buffer Zones' to Protect Fish Habitat in an Urban Environment" (MNR, 1987).

Table 3.4 Stream and Valley Corridor Buffer Strip Widths

Valley Buffer Width	
Stable slope	10 m from top of valley bank
Unstable slope (Protected toe of bank)	10 m setback from a stable slope angle (stable slope angle = 3:1 slope from toe of bank)
Unstable slope (Unprotected toe of bank)	100 year erosion + 10 m setback from a stable slope angle (100 year erosion = annual erosion rate × 100) (stable slope angle = 3:1 slope from toe of bank)
Stream Corridor Buffer Width	
Drainage Area > 125 ha	Regulatory Floodline + 10 m setback
Drainage Area < 125 ha	20 times the low flow channel width* + 10 m setback
Warm water fishery	15 m from the stream bank (2 year flow conditions)+
Cold water fishery	30 m from the stream bank (2 year flow conditions)+

* measured from the centre of the meander belt of the stream

\+ could also serve as a guide for determining the riparian buffer width. Alternative widths may be recommended based on site specific management objectives

3.5 Flow Splitters / By-Passes

Flow splitters are used to direct the runoff from a water quality storm into an end-of-pipe SWM facility, but bypass excess flows from larger events around the facility directly into the receiving waters. Since the MOEE/MNR Stormwater Quality Interim Guidelines (1989, 1990, 1991) have been released, the water quality storm in Ontario has generally been interpreted as a 13 mm (warm water fishery) or 25 mm (cold water fishery) storm.

Most Ontario municipalities will not accept mechanical/electrical controls on stormwater management facilities due to the potential for operational and maintenance problems associated with numerous real-time control systems within a municipality's jurisdiction. Therefore, the preferred flow splitter design operates on hydraulic principles.

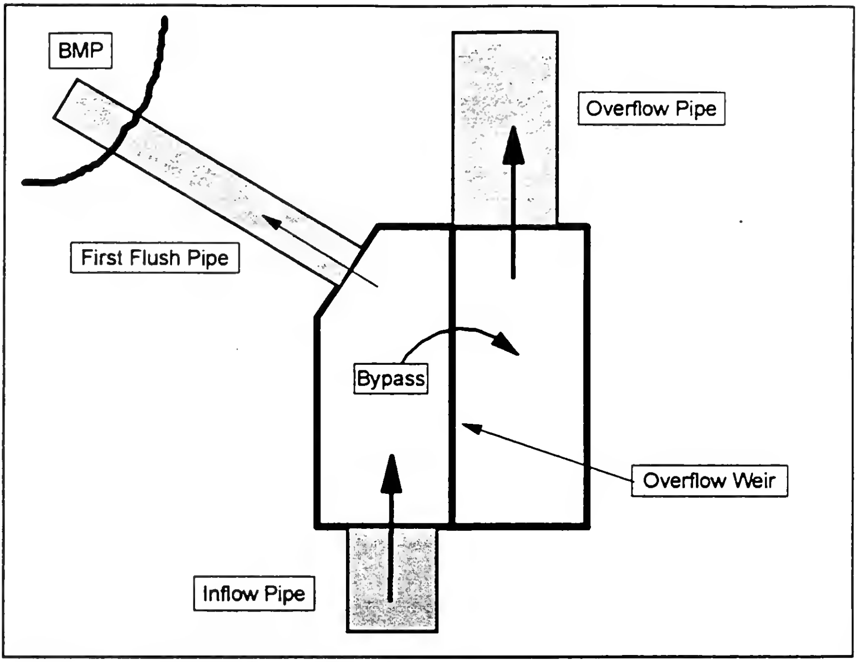


Figure 3.36 Storm Sewer Flow Splitter

The design of a hydraulically operated flow splitter must account for backwater conditions in the SWM facility, the hydraulic potential into the facility at the design bypass rate, and the potential for flow reversal during the recession limb of a storm.

Design Guidance

The typical flow splitter design is shown in Figure 3.36. Water is conveyed to the SWM facility via a first flush pipe. Once the facility reaches its design capacity, water backs up in the first flush pipe and hence, the flow splitter itself. Once the water level reaches the bypass elevation stormwater begins to bypass the SWM facility. The bypass is generally accommodated by a weir in the flow splitter structure.

The bypass elevation, bypass capacity, and first flush pipe capacity dictate how the flow splitter will operate.

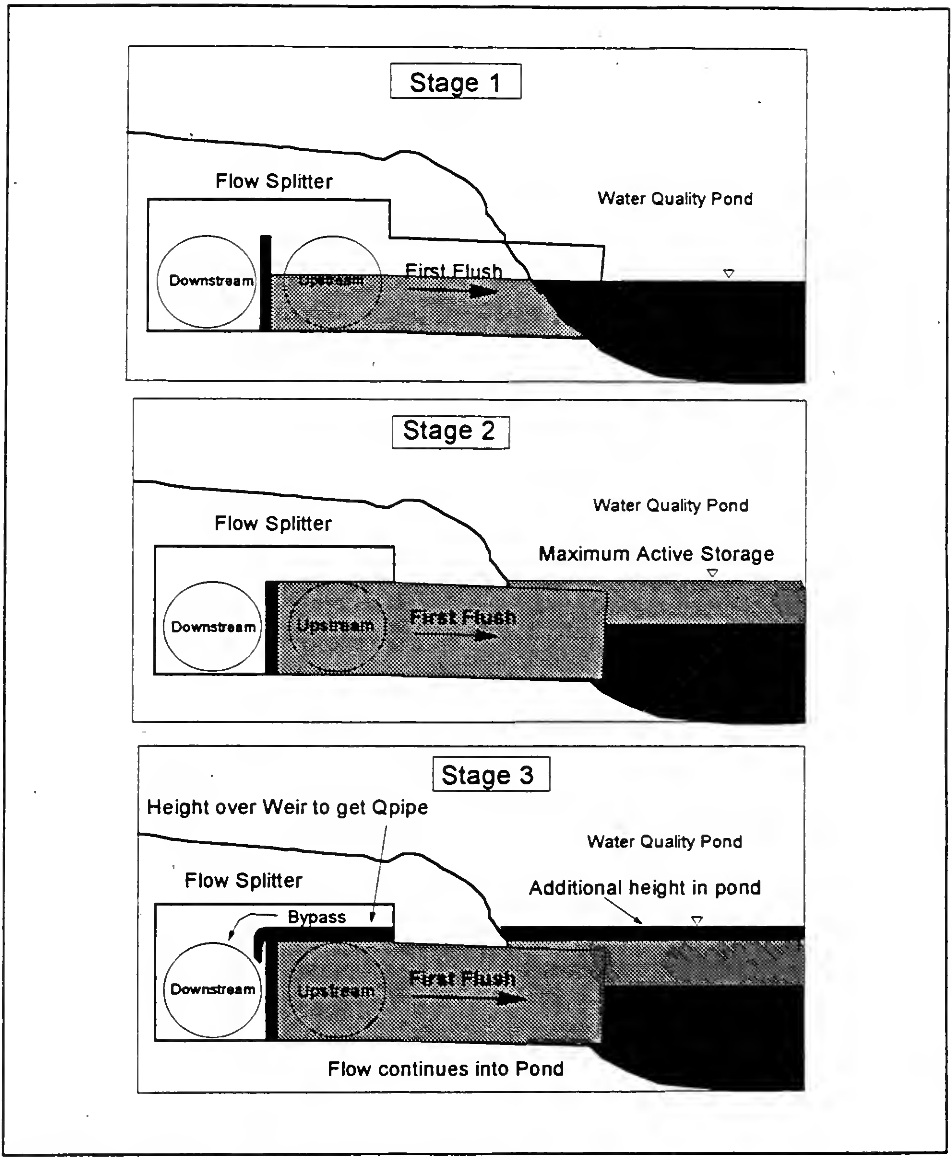


Figure 3.37 Operation of a Hydraulic Flow Splitter

Bypass elevation

There are two main methodologies for setting the bypass elevation. The first method is to set the bypass elevation such that the design storm is captured at the maximum bypass rate. Using this methodology the design water level in the end-of-pipe SWM facility would be equal to the depth of water in the bypass structure necessary to achieve the maximum anticipated bypass rate. Although this method ensures that the maximum design water level in the SWM facility is never exceeded, it also causes the bypass to operate for storms smaller than the 25 mm event.

The second methodology is to set the bypass elevation equal to the design storage elevation in the end-of-pipe SWM facility. Using this method the facility will only start to bypass once it has captured the design runoff volume. Although this ensures that the design volume will be captured before bypass occurs (given the first flush pipe capacity is not exceeded), it also means that the water level in the SWM facility will exceed the design level for large infrequent storms which utilize the bypass. This methodology is preferred since it is conservative. An example of the operation of this type of flow splitter is provided in Figure 3.37.

Bypass Capacity

Given that the bypass elevation is set equal to the design storage elevation in the end-of-pipe SWM facility, the maximum elevation in the facility depends on the rate of bypass with depth. For example, it would be ideal (from a hydraulic perspective) to have a long bypass weir, such that a small depth increment resulted in a large flow (and very little increase in depth in the facility). However, large weirs result in large flow splitter structures which are expensive. Therefore, an optimum balance between depth increase and bypass design should be sought recognizing that the increase in storage depth due to a flow splitter is not prolonged.

In end-of-pipe SWM facilities with only an extended detention outlet, the flow splitter has the potential to cause flow reversal. Flow reversal is the flow of water out of a SWM facility via the flow splitter structure. Flow reversal has the potential to occur when the flow over the bypass subsides quickly while the flow out of the pond is governed by a slow extended detention release. In these instances, water levels in the facility above the bypass elevation can force water back into the flow splitter and over the bypass structure during the recession limb of a bypass storm.

In order to minimize flow reversal several guidelines can be followed :

- provide an overflow outlet above the design water level in the SWM facility
- minimize the depth of bypass to achieve the desired bypass rate
- maximize the head losses between the facility and the bypass to minimize the hydraulic potential into the SWM facility

It should be recognized, however, that the provision of an overflow outlet will result in more water being conveyed into the pond, which defeats the original purpose of the flow splitter.

Therefore, the provision of an overflow outlet is not recommended as a standard practice to minimize flow reversal.

The operation of the by-pass must be assessed for the design event used to size the upstream pipe network. This event will vary from municipality to municipality (2 year, 5 year, 10 year storm). In some cases the upstream pipe network will be oversized due to overland flow constraints and may convey the 100 year storm. In all cases the splitter operation during the pipe design event must be assessed.

First Flush Capacity

The capacity of the first flush pipe into the SWM facility determines when a bypass occurs for intense storms, and it adds to the head loss between the bypass and the pond (this determines the hydraulic potential into the pond once the bypass begins to operate).

The design of the first flush pipe depends on the intensity of the water quality storm in question. Event simulations of a 2 hour Chicago distribution of the 25 mm storm produce peak flows which are similar to a 2 year storm (Note: a 25 mm storm can be a 100 year event if it occurs in a very short timeframe). Since the first flush is supposed to be representative of a frequent event, the 2 hour distribution is overly conservative. It is recommended that a 4 hour Chicago distribution of first flush storms be used to determine a peak flow for the sizing of the first flush pipe.

In pond systems, this will equate to the peak flow from a 25 mm storm. A 15 mm storm should be used for end-of-pipe infiltration systems and sand filters, and a 10 mm storm should be utilized for oil/grit separators and vegetated filter strips.

Maintenance By-Pass

In cases where the facility accommodates high flows as well as low flows, a flow splitter is not required. A maintenance by-pass is recommended, however, to facilitate maintenance operations in the urban SWMP.

The by-pass can be provided by a second pipe in a manhole upstream of the SWMP itself. The second pipe would be closed/blocked during the normal operation of the pond. During the maintenance periods, the inlet to the pond would be closed and the maintenance pipe opened.

The maintenance pipe could either discharge to an overland flow path around the SWMP, or to the outlet via an underground pipe system. If water is conveyed to the SWMP by swale, a separate maintenance swale must be constructed around the SWMP from the pond inlet, and the design must incorporate movable gates to re-direct flows into the maintenance swale during maintenance periods.

Technical Effectiveness

Flow splitters were introduced as a result of the concerns regarding re-suspension of settled pollutants in SWMPs during high flow events. The operation of hydraulic flow splitters depends on the characteristics of the runoff hydrograph. Flow splitters which are based on sizing a first flush pipe only are not optimum since the inflow pipe is based on a specific design storm (ie. it is likely that larger storms, with the same peak flow, will be conveyed to the facility).

Modelling which was performed in this study indicates that there is little difference between the long term effectiveness of facilities with and without flow splitters. The modelling, however, assumes that re-suspension does not occur. Recognizing the need to address re-suspension concerns, flow splitters are best utilized for SWMPs that cannot accept large volumes/flows and SWMPs that have a greater potential for re-suspension. As a general rule these SWMPs include:

- extended detention dry ponds (without a forebay)
- constructed wetlands (without a forebay)
- infiltration trenches/basins
- oil grit separators
- sand filters

3.6 Integration of Stormwater Management Objectives

The objective of stormwater management is to provide an integrated approach to water management (water quality, flooding, erosion, recharge), recognizing that stormwater management solutions must be economically efficient to construct and maintain.

Integration of the water management objectives has been individually discussed for each SWMP (in terms of appropriateness). A summary of guidelines and recommendations is provided in this section, however, to provide a better overview of the potential to integrate water management objectives with different urban SWMP types.

Table 3.5 provides a summary of the potential to address different water management objectives by urban development stormwater management practice. It should be re-iterated, however, that the process of selecting SWMPs is the cornerstone of providing an integrated approach to stormwater management.

- screen stormwater lot level controls for implementation
- assess stormwater conveyance controls for implementation
- implement end-of-pipe SWM facilities to address remaining concerns

Pervious pipes and pervious catch-basins are rated as both highly effective and having potential adverse effects. Although these systems will filtrate and adsorb pollutants by exfiltration into the surrounding native soil material, they also have the potential to contaminate the groundwater

since they convey road runoff which can be contaminated with hydrocarbons, spills, and chlorides from winter salting.

Table 3.6 provides guidance with respect to the integration of urban development SWMP design guidelines for water quality, flooding, and erosion control storage. Table 3.6 is expanded in greater detail in Sections 3.6.3, 3.6.4, and 3.6.5. **It should be noted that any reduction in storage requirements with respect to Table 4.1 represents a reduction in the volume of active storage required (not permanent pool).**

The cumulative effects of different SWMPs on temperature, spills control, and recharge is subjective, and is best assessed on a site-specific basis. Guidance from the subwatershed plan should be provided for temperature and recharge concerns. Additional guidance for recharge is provided in Section 3.6.1 and for temperature in Section 4.4.1.

3.6.1 Recharge / Infiltration Targets

The nature of the urban development SWMP selection process (ie. lot level, conveyance, end-of-pipe) should maximize the potential for recharge, and hence, satisfy recharge concerns. If the area is critical for infiltration/recharge it should be identified in the subwatershed plan, with targets specified for the stormwater management plan. In certain cases the subwatershed plan will exclude areas from the developable lands in the subwatershed.

If infiltration targets are specified in the subwatershed plan, the assumed soil conditions should be compared to the site specific soil conditions, and the infiltration target adjusted based on a comparison of the site specific infiltration potential to the assumed infiltration potential. Equation 3.10 provides a simple estimate of the adjusted infiltration requirements.

$$I = V \frac{P_{site}}{P_{SWP}}$$

Equation 3.10 Site Specific Infiltration Adjustment

where

I = Adjusted Infiltration Target (mm/ha)

V = Target Volume of Infiltration from subwatershed plan (mm/ha based on a specific storm event)

P_{site} = Percolation rate of site specific soils (dominant group) (mm/h)

P_{SWP} = Percolation rate of soils used in subwatershed plan (mm/h)

In areas where the percolation rate of the site specific soil is ≤ 15 mm/h, infiltration cannot be recommended on the site and 'I' should be set to zero. The use of Equation 3.10 for both soils with a higher percolation rate, and a lower percolation rate, will maintain the validity of the subwatershed plan.

Table 3.5 Stormwater Management Practice Potential						
SWMP	Water Quality	Flooding	Erosion	Recharge	Other	
					Temp.	Spills
Lot Grading	⊠	⊠	⊠	■	⊠	□
Roof Leader Ponding	⊠	⊠	⊠	■	⊠	□
Roof Leader Soakaway Pits	⊠	⊠	⊠	■	■	□
Pervious Pipes	■*	⊠	⊠	■	■	*
Pervious Catch-basins	■*	⊠	⊠	■	■	*
Wet Pond	■	■	■	□	*	■
Dry Pond	⊠	□	■	□	□	■
Dry Pond with forebay	■	■	■	□	□	■
Wetland	■	⊠	■	□	*	■
Wetland with forebay	■	■	■	□	*	■
Sand Filter	■	⊠	⊠	□	□	⊠
Infiltration Trench	⊠**	⊠	⊠	■	■	*
Infiltration Basin	⊠**	⊠	⊠	■	■	*
Vegetated Filter Strip	■	□	⊠	⊠	⊠	□
Buffer Strip	⊠	□	⊠	⊠	⊠	□
Oil/Grit Separator (offline or bypass)	⊠	□	□	□	□	■

■ highly effective (primary control)
⊠ limited effectiveness (secondary control)
□ not effective
* may have adverse effects
** effective pollutant removal (TSS, nutrients, metals, bacteria) but suspended solids removal reduces their longevity and hence effectiveness

Table 3.6 Urban Development Stormwater Management Practice Integration*

SWMP	Water Quality Benefits		Flooding Benefits	Erosion Benefits
	Event Modelling	Table 4.1	Event Modelling	Event Modelling
Lot Grading	increase pervious depression storage for lot areas by 0.5 mm for every 0.5% reduction in lot grade from 2%	water quality active storage equals (total area - $\sum$lot drainage areas) x Table 4.1 value + ($\sum$lot drainage areas x (Table 4.1 value - 5 m^3/ha for every 0.5% reduction in lot grade from 2%))	increase pervious depression storage for lot areas by 0.5 mm for every 0.5% reduction in lot grade from 2%	increase pervious depression storage for lot areas by 0.5 mm for every 0.5% reduction in lot grade from 2%
Roof Leader Ponding	model roof areas separately, increase impervious depression storage by (ponding volume/roof area)	water quality active storage equals (total area - $\sum$drainage area to ponding locations) x Table 4.1 value+, if ≥ 0, ($\sum$drainage area to ponding locations x Table 4.1 value - $\sum$ ponding volume (m^3))	model roof areas separately, increase impervious depression storage for roof areas by (ponding volume/roof area)	model roof areas separately, increase impervious depression storage for roof areas by (ponding volume/roof area)
Roof Leader Soakaway Pits	model roof areas separately increase impervious depression storage by (ponding volume/roof area)	water quality storage equals (total area - $\sum$roof area) x Table 4.1 value +, if ≥ 0, ($\sum$roof areas x Table 4.1 value - $\sum$ ponding volume (m^3))	model roof areas separately increase impervious depression storage by (ponding volume/roof area)	model roof areas separately increase impervious depression storage by (ponding volume/roof area)
Pervious Pipes	route flow through a DIVERT HYD or similar routing routine to divert exfiltrating flows based on orifice calculations. Exfiltrated flows should be routed through a reservoir based on feasible trench sizing with overflows added back into the pipe	water quality active storage equals total area x Table 4.1 value - pervious pipe storage	route flow through a DIVERT HYD or similar routing model to divert exfiltrating flows based on orifice calculations. Exfiltrated flows should be routed through a reservoir with overflows added back into the pipe	route flow through a DIVERT HYD or similar routing model to divert exfiltrating flows based on orifice calculations. Exfiltrated flows should be routed through a reservoir with overflows added back into the pipe
Pervious Catch-basins	route water quality event using a reservoir routine based on feasible trench sizing	water quality active storage equals total area x Table 4.1 value - pervious catch-basin storage	route water quantity events using a reservoir routine based on feasible trench sizing and evaluate incidental benefits	route erosion event using a reservoir routine based on feasible trench sizing and evaluate incidental benefits
Enhanced Grass Swales	route water quality event using a reservoir routine	water quality active storage equals total area x Table 4.1 value - enhanced grass swale storage	route water quantity events using a reservoir routine based on enhanced grass swale design	route erosion event using a reservoir routine based on enhanced grass swale design
Wet Pond	route water quality event using a reservoir routine and size storage to obtain desired detention time	directly calculated using Table 4.1	route quantity events with erosion/quality reservoir routine and size quantity storage	route erosion event using water quality reservoir routine and size erosion storage

Table 3.6 Urban Development Stormwater Management Practice Integration (continued)

SWMP	Water Quality Benefits		Flooding Benefits	Erosion Benefits
	Event Modelling	Table 4.1	Event Modelling	Event Modelling
Dry Pond	route water quality event using a reservoir routine and size storage to obtain desired detention time	directly calculated using Table 4.1	size dry pond based on quality and evaluate incidental benefit during quantity storms	route erosion event using water quality reservoir routine and size erosion storage
Dry Pond with forebay	route water quality event using a reservoir routine and size storage to obtain desired detention time	directly calculated using Table 4.1	route quantity events with erosion/quality reservoir routine and size quantity storage	route erosion event using water quality reservoir routine and size erosion storage
Wetland	route water quality event using a reservoir routine and size storage to obtain desired detention time	directly calculated using Table 4.1	size wetland based on quality and evaluate incidental benefit during quantity storms	route erosion event using water quality reservoir routine and size erosion storage
Wetland with Forebay	route quality event using a reservoir routine and size storage to obtain desired detention time	directly calculated using Table 4.1	route quantity events with erosion/quality reservoir routine and size quantity storage	route erosion event with quality reservoir routine and size erosion storage
Sand Filter	route water quality event using a reservoir routine based and size storage to prevent overflows	directly calculated using Table 4.1	size filter based on quality and evaluate incidental benefits from filter during quantity storms	route erosion event using water quality reservoir routine and size erosion storage
Infiltration Trench	route water quality event using a reservoir routine based on feasible trench sizing	directly calculated using Table 4.1	size trench based on feasibility and evaluate incidental benefits from trench during quantity storms	size trench based on feasibility and evaluate incidental benefits from trench during erosion storm
Infiltration Basin	route water quality event using a reservoir routine based on feasible basin sizing	directly calculated using Table 4.1	size basin based on feasibility and evaluate incidental benefits from basin during quantity storms	route erosion event using water quality reservoir routine and size erosion storage
Vegetated Filter Strip	route water quality event using a reservoir routine with a weir outlet to size storage based on the allowable weir flow depth	NA	NA	size filter strip based on water quality event and evaluate incidental benefits during the erosion storm
Buffer Strip	NA	NA	NA	NA
Oil/Grit Separator (3 chamber or manhole)	NA	NA	NA	NA

* the storage provided by stormwater lot level and conveyance controls may be reduced by a longevity factor (see Sections 3.6.3, 3.6.4, and 3.6.5) when assessing end-of-pipe SWM facility storage requirements for water quality, quantity and erosion.

3.6.2 Spills Control

Spills control is related to land use (industrial, commercial) and should be implemented as an additional requirement to the water quality concerns.

3.6.3 Stormwater Lot Level Controls

Lot Grading

Modelling - Water Quality, Erosion and Quantity Control

The effect of lot grading on water quality, erosion, and flooding depends on the extent of grading changes. Recommended lot grading changes include the provision for a typical standard slope (2 - 5%) within 2 - 4 metres of any buildings and a flatter slope (<2%) for the remaining lot area. These grading changes are only appropriate if the site is naturally flat.

The water management benefit derived from these grading changes given a typical lot (12 m frontage by 30 m depth) is estimated to be an additional 0.5 mm of pervious depression storage for every 0.5% reduction in the typical 2% grading standard (ie. 1.5% grade = +0.5 mm, 1.0% grade = + 1 mm, 0.5% grade = +1.5 mm).

The additional depression storage can be further adjusted based on a "longevity factor" which takes into account public acceptance and landowner alterations to individual site grading (Equation 3.11). It is recommended that a longevity factor of 0.75 be used for developments that implement flatter lot grading.

$$DSP = 4.67 + (2 - G)\,f$$

Equation 3.11 Pervious Depression Storage Adjustment

where :
- DSP = pervious depression storage (mm)
- G = lot grading (%)
- f = longevity factor

A higher or lower factor could be used if requested by a regulatory agency (municipality, MNR, MOEE, CA). This request would be made based on the agency experience with people altering site grading patterns. Currently, there is a tendency for people to make minor alterations to their lot grading.

Event modelling using the altered pervious depression storage for areas with flatter lot grading will account for the effect of the grading on the required end-of-pipe water quality, quantity, and erosion storage.

Table 4.1 - Water Quality

The effects of flatter lot grading on the water quality storage requirements presented in Table 4.1 can be estimated by Equation 3.12.

$$V = (A\text{-}LL)\times S + LL\times[S-((2-G)\times 5\times f)]$$ Equation 3.12 Lot Grading Adjustment

where V = volume of water quality storage required (m^3)
A = total development area (ha)
LL = development area with flat lot grading (ha)
S = original water quality requirement from Table 4.1 (m^3/ha)
G = flatter lot grades (%)
f = longevity factor

If the second term on the right hand side of Equation 3.12 is negative (ie. more storage is provided by the flatter lot grading than is required) it should be set to zero. The development area with flatter lot grading (LL) should not include the road right-of-ways.

Roof Leader Discharges to Surface Ponding Areas

Modelling - Water Quality, Erosion and Quantity Control

The benefits from discharging roof leaders to on-site surface ponding areas on water quality, erosion, and flooding can be estimated by altering the impervious depression storage for rooftop areas.

The change in impervious depression storage depends on the volume of surface ponding provided and the roof area being controlled. Equation 3.13 provides an estimation of the adjusted impervious depression storage. The use of Equation 3.13 implies that the roof areas will be modelled separately since the impervious depression storage of driveways and roads should not be altered.

$$DSI = \frac{SPV}{RA} \times 1000 \times f$$ Equation 3.13 Impervious Depression Storage

where DSI = adjusted impervious depression storage (mm)
SPV = volume of surface ponding storage (m^3)
RA = rooftop area (m^2)
f = longevity factor

Table 4.1 - Water Quality

The effects of surface storage for roof leader discharges on the water quality storage

requirements presented in Table 4.1 can be estimated by Equation 3.14.

$V=(A-RS)\times S+(RS\times S)-(SPV\times f)$ Equation 3.14 Roof Leader Surface Storage Adjustment

where V = volume of water quality storage required (m^3)
A = total development area (ha)
RS = rooftop area + surface ponding area (m^2)
S = water quality requirement from Table 4.1 (m^3/ha)
SPV = volume of surface ponding storage (m^3)
f = longevity factor

If the second term on the right hand side of Equation 3.14 is negative (ie. more surface storage is provided than is required) it should set to zero. The rooftop area as defined in Equation 3.14 includes the surface storage area (and surface drainage tributary to the ponding area).

The longevity factor in Equation 3.14 accounts for the public acceptance of local ponding areas on their property. It is recommended that a longevity factor of 0.75 be used for surface ponding storage for rooftop leader discharges.

Roof Leader Discharges to Soakaway Pits

Modelling - Water Quality, Erosion and Quantity Control

The overall water management benefits from the implementation of soakaway pits can be calculated using Equation 3.13 and Equation 3.14. The term SPV (surface ponding volume) would be replaced with the volume of void space in the soakaway pit (eg. 40% of the gravel storage volume with 50 cm gravel).

Another method to calculate the benefit of the soakaway pits with respect to water quality, erosion, and quantity control is to model them as reservoirs. The areas with soakaway pits would be lumped together and modelled separately. The flow from this area would be routed through a reservoir. The outflow rate from the reservoir would depend on the configuration of the soakaway pit and the volume of water in the soakaway pit. A rating curve can be calculated based on the storage and flowrate as shown in Equation 3.15. It is recommended that the longevity factor be based on the percolation rate of the surrounding soils due to the lack of widespread experience with the implementation of soakaway pits. Table 3.7 provides recommended longevity factors based on native soil percolation rates.

$Q = f\times(P\div 3{,}600{,}000)\times(2LD+2WD+LW)\times n$ Equation 3.15 Soakaway Pit Rating Curve

$V = LWD \times n \times f$ (Rating Curve Storage Volume)

where Q = flowrate (m^3/s) for a given storage volume (V)

f	=	longevity factor
P	=	native soil percolation rate (mm/h)
L	=	length of the soakaway pit (m)
W	=	width of the soakaway pit (m)
D	=	depth of water in the soakaway pit (m)
V	=	volume of water in the soakaway pit (m^3)
n	=	void space in the soakaway pit storage layer

Table 3.7 Longevity Factors for Conveyance Media

Media Percolation Rate (mm/h)	Longevity Factor (f)
< 25	0.5
> 25 but < 100	0.75
> 100	1.0

3.6.4 Stormwater Conveyance Controls

Pervious Pipes

Modelling - Water Quality, Erosion and Quantity Control

The benefits provided by the implementation of pervious pipes with respect to water quality, erosion, and quantity storage are more difficult to quantify since the exfiltration is dependent on many factors (pipe slope, number of perforations, size of perforations, depth of flow). Recent work (Wisner and Associates, 1993) has indicated that the exfiltration can be reasonably modelled using the orifice equation with a variable orifice coefficient. The orifice coefficient varies from 0 to a maximum of 0.63 and is dependent on both the perforation size and the depth of flow. However, this calculation must be performed along the pipe as water flows through the system since water is continuously exfiltrating as it moves through the pipe.

The results from the Wisner study were used to derive a steady state equation for the rate of exfiltration with flow through the pipe. Although, this simplified relationship is less accurate than continuous simulation it provides a reasonable approximation of the pipe flow - exfiltration relationship. The exfiltration rate for any given flow rate in a pipe can be approximated using Equation 3.16. This equation indicates that the exfiltration rate of exfiltration depends on the inflow rate, the size and number of perforations along the pipe, and the slope of the pipe. The results for Equation 3.16 were compared to the laboratory tests performed in the above-noted study. Equation 3.16 provides a reasonable estimation of the measured flows as shown in

Appendix L. A derivation of Equation 3.16 is also provided in Appendix L.

$$Q_{exft} = (15A - 0.06S + 0.33)\ Q_{inf}$$ Equation 3.16 Exfiltration Discharge

where Q_{exft} = exfiltration flow through pipe perforations (m^3/s)
A = area of perforations per metre length of pipe (m^2/m)
S = slope of perforated pipe (%)
Q_{inf} = flow through the perforated pipe (m^3/s) (longitudinally)

Equation 3.16 was developed based on the measured flows in a 300 mm diameter pipe with 12.7 mm and 7.9 mm perforations. The use of Equation 3.16 for larger diameter pipes, or pipes with much larger perforations should be scrutinized for reasonableness.

Based on equations 3.16, a rating curve can be estimated for the exfiltration flow as a function of flow in the perforated pipe system. Using this relationship a hydrograph diversion routine (such as DIVERT HYD in OTTHYMO 89) can be used to determine the split in flows that are conveyed by the perforated pipe system.

The diverted flows that represent exfiltration should be further routed through a reservoir routine based on the exfiltration storage, and native material. Equation 3.15 can be used to determine the routing reservoir rating curve for the exfiltrated water. The term 2WD in Equation 3.15 should be set to zero in this application since the exfiltration storage is a linear system. The overflows from the reservoir signify that the exfiltration storage is full, and should be added back to the pipe flow. This can be accomplished using a second DIVERT HYD command (OTTHYMO 89). This methodology will account for the benefits of the perforated pipe system during erosion events, quantity events, and water quality events.

Table 4.1 - Water Quality

If Table 4.1 is used to determine the necessary water quality control, the volume of exfiltration storage can be compared directly with the infiltration storage required to achieve the desired level of water quality protection for the appropriate level of imperviousness. The required infiltration storage should be adjusted based on the longevity factor used.

A longevity factor based on Table 3.7 is recommended for use in the determination of perforated pipe benefits.

Pervious Catch Basins

Modelling - Water Quality, Erosion, and Quantity Control

The benefit with respect to water quality, quantity, and erosion control resulting from the installation of pervious catch basins can be estimated using the same methodology as that derived for soakaway pits (ie. reservoir routing using Equation 3.15). The modelling of pervious catch-basins is easier than soakaway pits since the lot and road areas do not have to be separated into different basins.

Table 4.1 - Water Quality

The effects of pervious catch-basins on the water quality storage requirements presented in Table 4.1 can be estimated by Equation 3.17.

$$V = (A \times S) - (CBV \times f)$$ Equation 3.17 Pervious Catch-Basin Adjustment

where	V	=	volume of water quality storage required (m^3)
	A	=	development area draining to pervious catch-basins (ha)
	S	=	water quality requirement from Table 4.1 (m^3/ha)
	CBV	=	volume of pervious catch-basin storage (m^3)
	f	=	longevity factor

The longevity factor should be estimated from Table 3.7.

Enhanced Grass Swales

Modelling - Water Quality, Erosion, and Quantity Control

Enhanced grass swales have a permanent check dam to hold back water during small events. The check dam acts as a weir during larger events and can be modelled as a reservoir. The rating curve for the reservoir can be determined based on the stage storage relationship upstream of the check dam in the swale and the weir equation (Equation 3.9). Given the small storage volume contained in one swale, and the likelihood for numerous swale areas, the storage volume from several swales should be lumped together in this assessment (ie. downstream check dam controls lumped storage). In order to ensure that numerical instability does not occur in the routing routine, there should be a positive discharge from the swale as the storage increases (even below the elevation of the check dam). The discharge from the swale below the elevation of the check dam can be calculated using Equation 3.18 where the term LW represents the contact area between the water and the swale (ie. the wetted perimeter of the swale below the check dam). The percolation rate (P) in Equation 3.18 should be assessed for the native soil material and the porosity should be set to 1. The longevity factor for enhanced grass swales should be 1.0 since they are not directly dependent on infiltration for operational performance.

Table 4.1 - Water Quality

The effects of enhanced grass swales on the water quality storage requirements presented in Table 4.1 can be estimated by Equation 3.17. The term CBV would be replaced with the storage provided upstream of the enhanced grass swale at the elevation of the check dam.

3.6.5 End-of-Pipe SWM Facilities

Wet Pond

The benefits with respect to erosion and flooding from the implementation of water quality storage in a wet pond are easily calculated. The required water quality storage (either from Table 4.1 or the Subwatershed Plan) can be modelled as a storage reservoir in an event model. The values in Table 4.1 are based on a 24 hour detention. Additional points can be added to the reservoir to simulate erosion control (if different from water quality control) and flood control. This methodology can be used if quality, quantity, and erosion are accommodated in the one pond. Multiple reservoir routines can be implemented for separate facilities. Figure 3.38 illustrates the zones in a combined quality and quantity wet pond.

Dry Pond

A dry pond should not be used for combined quality and quantity control unless a forebay is included in the design due to the potential for re-suspension and scour of previously settled pollutants. The benefits with respect to erosion and flooding from the implementation of water quality storage in a dry pond which incorporates a forebay are calculated in the same manner as for a wet pond.

Wetland

A wetland should not be used for combined quality and quantity control unless a forebay is included in the design due to the potential for re-suspension and scour of previously settled pollutants. The benefits with respect to erosion and flooding from the implementation of water quality storage in a wetland which incorporates a forebay are calculated in the same manner as for a wet pond.

Sand Filter

The sizing of a sand filter for water quality control is based on the runoff to be treated (either from Table 4.1 or the runoff from a 15 mm storm) and the filtration area. The outflow from the sand filter should be based on Equation 3.18.

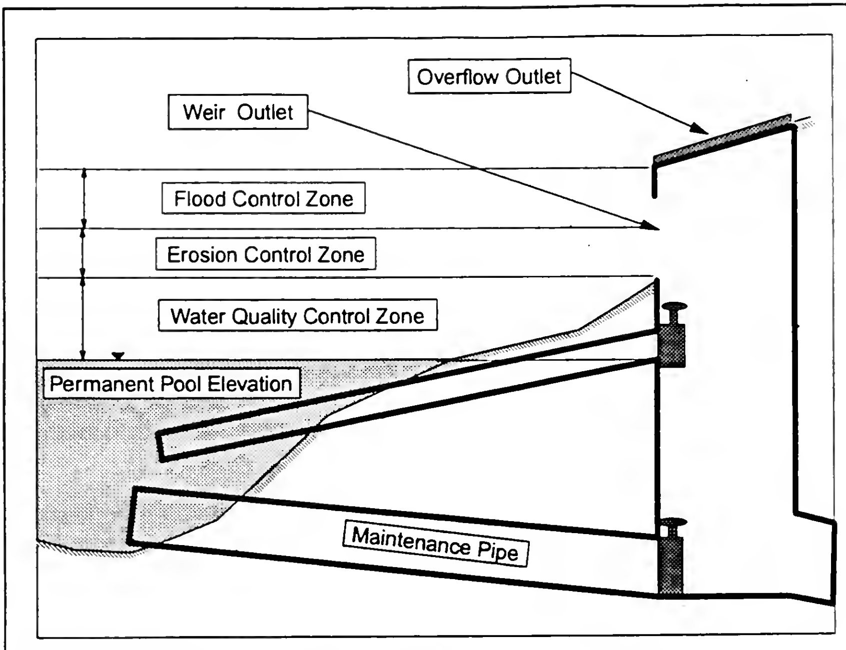


Figure 3.38 Multiple Objective Pond Outlet

$Q = f \times (P \div 3{,}600{,}000) \times (LW \times n)$ Equation 3.18 Sand Filter Discharge

where Q = flowrate (m^3/s) out of the sand filter
f = longevity factor
P = percolation rate for sand (mm/h)
L = length of the filter (m)
W = width of the filter (m)
n = void space in the sand filter (typically 0.25)

Only the length and width (bottom area) are used in Equation 3.18 since the assumption is made that the filter is contained within an impermeable liner causing all of the discharge to be collected in the perforated pipes at the bottom of the filter.

The longevity factor in Equation 3.18 should be based on the percolation rate of the sand used from Table 3.7.

In order to assess the incidental benefits of sand filters in reducing erosion and flooding storage

requirements, the sand filters should be modelled using a reservoir routine. An assumption of constant outflow with increasing storage can be made as a conservative assumption. Some marginal increase in flow will have to be assumed with increasing storage recognizing the mathematical instabilities inherent in most reservoir routines. The rating curve above the water quality storage volume should reflect conveyance without attenuation to simulate the by-passing of the filter.

Infiltration Trench

The sizing of an infiltration trench is based on the runoff to be treated (either from Table 4.1 or the runoff from a 15 mm storm) recognizing that a lower size may be implemented based on the physical constraints imposed by the site. The same methodology used for sand filters to account for incidental flooding and erosion benefits can be used for infiltration trenches. The assumption of using the bottom area of the trench is still reasonable since the infiltration trench has a sand filter layer for pollutant removal (ie. the trench should be designed to convey water through the bottom).

The percolation rate (P), and longevity factor (f) in Equation 3.18 should be assessed for the native material below the infiltration trench recognizing that the surrounding native material will be the limiting conveyance media. The porosity in Equation 3.18 should be equal to the void space in the storage media (0.4 for clear stone). The rating curve above the water quality storage volume should reflect conveyance without attenuation to simulate the by-passing of the trench.

Infiltration Basin

The assessment of incidental flooding and erosion control benefits derived from the implementation of an infiltration basin is identical to that for an infiltration trench. The bottom area of the basin in Equation 3.18 represents the contact area of water and the basin floor. As such, a reservoir routing rating curve can be developed based on the grading in the basin (ie. stage-storage-contact area relationship) and Equation 3.18 (with the porosity set equal to 1). The rating curve above the water quality storage volume should reflect conveyance without attenuation to simulate the by-passing of the basin.

Vegetated Filter Strip

The sizing of the level spreader is based on the conveyance of the peak flowrate from a 10 mm (4 hour Chicago storm) with a depth of less than 100 mm based on the weir equation (Equation 3.9). If a short spreader is utilized there may be a need for storage upstream of the level spreader to ensure that flow depths through the filter strip do not exceed 100 mm. A reservoir routine can be used to assess the water quality storage required upstream of the level spreader.

If there is appreciable storage designed upstream of the level spreader the incidental flood and erosion control benefits from the filter strip can be assessed using the reservoir routine (The filter strip should not be designed specifically for erosion and flood control however). In most cases, however, it is expected that the level spreader will be designed such that there is minimal upstream storage required. It should be assumed that there are no flood and erosion control benefits resulting from the implementation of a filter strip under these conditions.

Buffer Strip

As previously mentioned buffer strips provide limited benefits in terms of water quality enhancement, and flood and erosion control (unless of flow from the development is conveyed as sheet flow across the buffer strip). The main reasons for implementing buffer strips are oriented more towards valley corridor protection, wildlife migration, natural area linkages, etc. Therefore, it is recommended that buffer strips not be assessed in terms of water quality, erosion, and quantity benefits.

Oil/Grit Separators

Oil/grit separators do not provide significant stormwater quality control benefits based on existing monitoring studies. Oil/grit separators do not provide extended detention control and hence do not provide erosion or flood control benefits. Oil/grit separators are best implemented for spills control based on the proposed land use. It is recommended that oil/grit separators not be assessed in terms of water quality, erosion, and quantity benefits.

3.7 Major System Flows

The emphasis placed on source level controls for water management by this manual should not overshadow major system flowpath requirements. The implementation of source controls makes the design of the major system flow path more important.

Overland flow paths must be considered in the design of facility overflows/by-passes (sand filters, infiltration trenches, infiltration basins, soakaway pits, lot level ponding), and stormwater conveyance controls (enhanced grass swales).

The major system should be designed with sufficient capacity to convey flows assuming that all of the source controls and stormwater conveyance controls have failed (ie. no soakaway pit storage, grassed swales full of water up to the check dams).

3.8 Watercourse Alterations

In all development submissions there should be a concerted effort to preserve the existing watercourse in its natural state. Alterations to the watercourse may be required in certain instances however (ie. in areas with little topographical relief and urban (storm sewers) servicing, and in areas where there are proposed stream crossings (culverts, bridges, etc.).

In cases where watercourse alterations are required, the alterations should be designed using natural channel design techniques. This will require that the current stream reach be assessed using a stream classification system and that the any alterations must account for hydrologic and hydraulic changes as a result of urbanization and the preferred stream function/morphology. Guidance on natural channel design techniques is provided in "Natural Channel Systems : An Approach to Management and Design - Development Draft" (Ministry of Natural Resources, 1994).

3.9 Temperature Mitigation Measures

There are a number of reports which indicate that urban development end-of-pipe SWM facilities increase the temperature of water before it is discharged to the receiving waters (Beland, 1990, Galli, 1991, Schueler, 1992). These reports also stress, however, that an increase in water temperature is inevitable if an area is developed (ie. urbanization causes stormwater temperature increases). This observation is based on current development practices. It is anticipated that the temperature increase due to urbanization can be minimized if the subdivision/site planning techniques, which were discussed in Chapter 2, are implemented.

Wet ponds, and wetlands can compound the temperature increase due to urbanization by maintaining water in the facility between storms and allowing it to acclimate to the air temperature. (It is interesting to note that Galli (1991) found that there is an increase in water temperature with all types of urban development SWMPs).

There are several techniques that can be used to reduce the thermal impacts from wet ponds :

- Pond Configuration

  The configuration of the pond will affect the temperature in the pond. The length to width ratio should be maximized to prevent the occurrence of large open areas of water which cannot be shaded by riparian vegetation. Planted berms and islands can be implemented in ponds that have a poor configuration due to the site specific topography/land available.

- Riparian Planting Strategy

  Planting in the shoreline fringe and flood fringe zones of a wet pond help to shade the pond and minimize temperature increases during inter-event

periods. The planting strategy should incorporate designs which shade open water areas when the vegetation reaches maturity.

- Bottom-draw Outlet

  There are temperature benefits from a bottom draw facility, although this is dependent on the size of the permanent pool and the release depth.

  There is a minimal difference in temperature of water within the top metre of the permanent pool. Greater temperature decreases (in the order of several degrees Celsius) occur several metres below the permanent pool surface ($\geq$ 2 metres). The lowest water temperatures occur in water greater than 3 metres in depth. Ponds with permanent pool depths greater than 3 metres, however, are likely to become thermally stratified during the summer months. The water at this depth can become anoxic, and there is the potential for metals and nutrients which previously had settled out to come back into re-suspension. Although the oxygen deprivation can be solved by re-aeration at the outlet (discharge over rocks, etc.) the discharge of the polluted water would be undesirable. Accordingly, ponds with a very deep release (> 3 m) should consider the use of a re-aeration fountain in the pond itself to prevent thermal stratification from occurring.

- Subsurface Trench Outlet

  Treatment of water at the discharge point from a pond has also been suggested to cool the water. Generally these systems discharge water from the pond through a subsurface trench filled with clear stone. These systems use the mechanism of heat transfer from the water to the stone as it flows through the trench. These systems do not rely on infiltration and are purely conveyance systems. There is relatively little experience with respect to the success of these systems.

  The length, width, and depth of the system depends on the intended range of release rates, and the proximity of the pond to the watercourse. The cross-sectional area of the trench should be sized based on the design conveyance flow. Additional area will not be utilized and is therefore ineffective. The length of the trench should be maximized such that the contact of rock and water is maximized.

  The design conveyance through the subsurface trench does not necessarily have to match the design release rate from the pond (especially if the pond will accommodate the runoff from relatively large storms - ie. 25 mm). These systems should be designed to mitigate the temperature for the everyday storm (ie. $\leq$ 10 mm) since the temperature impacts from infrequent events are less important.

The trench should be wrapped with non-woven filter fabric to prevent the native material from blocking the pore space in the stone/rock. The stone should be relatively small (13 mm - 25 mm) since smaller stone will have a greater total surface area available for heat transfer.

- Night Time Release

  Monitoring evidence (Beland, 1990 unpublished) suggests that the water in stormwater ponds cools during the night as a result of the influence of the ambient temperature. This temperature decrease is related to the ambient temperature fluctuation and can be up to 5 °C. Generally, the lowest pond temperatures were recorded during the early morning (5 a.m. - 7 a.m.) indicating that very early morning releases should be targeted for facilities which are designed with real time controls. Although the pond temperature at this time is still elevated above the ambient temperature, the impacts to the receiving waters will be minimized since the water is not superheated by the solar radiation and higher daytime temperatures.

3.10 Outlet Channel Design

In cases where there is a lengthy outlet channel from the end-of-pipe SWM facility to the receiving waters, natural channel design techniques can be employed. Guidance on natural channel design techniques is provided in "Natural Channel Systems : An Approach to Channel Management and Design - Development Draft" (Ministry of Natural Resources, 1994).

The outlet channel from an end-of-pipe SWM facility to the receiving waters should be shaded by plantings to minimize the temperature of water discharged to the receiver.

3.11 Winter Impact on Urban Development SWMP Performance

Most urban development SWMPs will not be effective during the winter period. Small ponds/wetlands will tend to freeze and any vegetation in these facilities, or vegetative SWMPs themselves, will be ineffective during this period of time.

Subsurface systems such as perforated pipes, pervious catch-basins, and deep trenches/soakaway pits will still be operational during the winter, however, the effectiveness of these stormwater conveyance measures during the spring when the soils are saturated is expected to be reduced.

The operation of infiltration systems that accept road runoff (ie. perforated pipe systems, pervious catch-basins) during the winter is questionable given the elevated suspended solids loads due to the sanding of the roads, and elevated chloride loads due to the salting of the roads. Consideration should be given to the implementation of a by-pass system for winter operation

of these systems, or the assessment of these measures at a reduced longevity factor (ie. shorter life span) to reflect the higher potential for clogging during winter/spring operation.

The reduction in water quality performance of stormwater management measures during the winter period is not regarded as being significant since there is a reduced volume of stormwater in the winter. The reduction in water quality performance during the spring period is accepted since there are large volumes of stormwater during this period which are considered to be economically infeasible to treat. The large volume of stormwater during the spring period is also considered to be a source of dilution reducing the concerns regarding instream habitat degradation. Rightly, or wrongly, the impact on basin export during this period (ie. export of pollution to the Great Lakes) cannot be addressed using natural enhancement techniques due to the physical constraints imposed by the Ontario climate.

4.0 SWMP SELECTION AND DESIGN IN THE ABSENCE OF SUBWATERSHED PLANNING

4.1 General

Urban development without watershed/subwatershed planning is generally discouraged since there are many environmental impacts that cannot be addressed at a plan of subdivision or site plan :

- cumulative impact of urbanization on aquatic resources
- wildlife corridors
- natural area linkages
- rehabilitation areas
- cumulative impact of individual subdivision/site water management practices

One of the goals of watershed/subwatershed planning and land use planning, in areas which are predominantly undeveloped, is to define where development should occur. Accordingly, development planning in the absence of subwatershed planning input to land use planning is inappropriate.

As indicated above, development planning in the absence of subwatershed planning cannot address cumulative impacts. The cumulative impacts of development refer to the joint effect of numerous single developments on the downstream watercourse. These impacts relate to :

- flooding
- erosion
- temperature
- baseflow
- nutrient enrichment in a lake
- bacteria loading to a beach

Although development planning through subwatershed planning is preferred, there will be many cases where a development will be allowed to proceed without a subwatershed plan. In these cases, the proposed development may be an infill situation (surrounded by existing development), or a replacement for existing development, or an expansion of the urban fringe. In most of the cases where a development is allowed to proceed without subwatershed planning, the preparation of a subwatershed plan is determined to be cost ineffective (since there is very little foreseen future development pressure) or cost prohibitive in the near future. The remainder of this chapter outlines a methodology for stormwater management planning and design in these situations. This methodology consists of 4 steps :

1. Subdivision / Site Planning
2. Assessment of Receiving Water Concerns

3. Selection of Water Management Criteria
4. Urban SWMP Selection

4.2 Subdivision / Site Planning

In the absence of watershed/subwatershed planning, subdivision/site planning must occur to ensure that the development is planned with due regard to the surrounding environment. Resource mapping must be prepared to ensure that the development does not impact significant natural areas since there will not be any commensurate mapping from a subwatershed plan. Accordingly, the methodology recommended in Chapter 2 should be followed to produce a development layout that protects the natural function of the land and reserves areas for stormwater management.

4.3 Assessment of Receiving Water Concerns

The second step in the formulation of water management solutions without planning information is to identify the receiving water concerns. These concerns can be classified into 4 groups of general criteria :

water quality	aquatic habitat, pollutant loading, recreation (swimming, boating)
water quantity	flooding
erosion potential	in-stream erosion
baseflow	groundwater recharge, in-stream baseflow/low flow maintenance

It should be recognized that all of these concerns/criteria may be affected by land use change. They are addressed in order to ensure that the receiving waters can support an appropriate diversity of life and will not undergo undesirable geomorphologic change.

4.4 Selection of Water Management Criteria

4.4.1 Water quality

As part of this study, a review of the existing water quality criteria were made. The 1991 MOEE/MNR Interim Stormwater Quality Control Guidelines for New Developments required that the runoff from a 13 mm or 25 mm storm be detained for 24 hours (13 mm for a warm water fishery, and 25 mm for a cold water fishery).

Although the 1991 MOEE/MNR Interim Guidelines have been instrumental in addressing water quality concerns over the past 5 years, it has always been recognized that these criteria would evolve as the understanding of stormwater quality control evolved.

In an effort to refine these criteria, computer modelling of end-of-pipe stormwater management facilities was undertaken to determine the variation in pollutant removal with SWMP type and level of imperviousness. A continuous simulation model of urban development was established using the SWMM model for four levels of imperviousness :

- 35 % imperviousness
- 55 % imperviousness
- 70 % imperviousness
- 85 % imperviousness

The build-up and wash-off of suspended solids was modelled in these four scenarios as input to the end-of-pipe SWM facilities. Suspended solids were modelled since they are a primary concern in terms of lethality and chronic effects. Nutrients, metals, and oil/grease were not modelled since they are considered secondary concerns which predominantly result in chronic effects. In addition, nutrients, metals, bacteria, and oil/grease are generally regarded as sediment associated indicating that the effectiveness of an end-of-pipe SWMP with respect to these parameters is proportional to the suspended solids removal results.

Temperature and dissolved oxygen were not modelled since they are instream concerns (not pollutant export concerns) which are governed by the site specifics of a subwatershed. A subwatershed plan should be undertaken to assess these parameters if they are of significant concern.

A continuous simulation program was developed to estimate suspended solids settling in various types of end-of-pipe stormwater management facilities :

- wet ponds
- dry ponds
- infiltration systems
- wetlands

Two types of dry ponds were simulated (continuous and batch operation). Continuous operation assumed a hydraulic outlet which released water based on the volume of water in the facility. The batch control operation assumed that the outlet was closed at the beginning of runoff into the pond, and then opened 24 hours later to release the stored water.

The settling model divided the suspended solids loading into six particle sizes based on the distribution provided in Table 3.3. The removal of each particle size was based on either dynamic or quiescent settling. The determination of the settling conditions in the SWMP (ie. dynamic or quiescent) was dependent on the volume of water influent to the facility in a certain timestep compared to the volume of water existing in the facility. The SWMM model, the settling model, and the modelling methodology are described in detail in Appendix H.

This type of modelling was initially undertaken for 14 different meteorological stations across

the province to determine if there were regional variations in suspended solids settling performance. The analysis timeframe varied for each station according to the period of record for the station. Estimations of the suspended solids settling for the 14 different meteorological stations were also made using a statistical model developed at the University of Toronto (Guo, 1991) for comparison with the continuous simulation results.

The results from the regional analysis are shown in Figure 4.1. Figure 4.1 indicates that there are no significant regional differences in suspended solids removal across the province (There were variations in the order of 5% which are well within the modelling error).

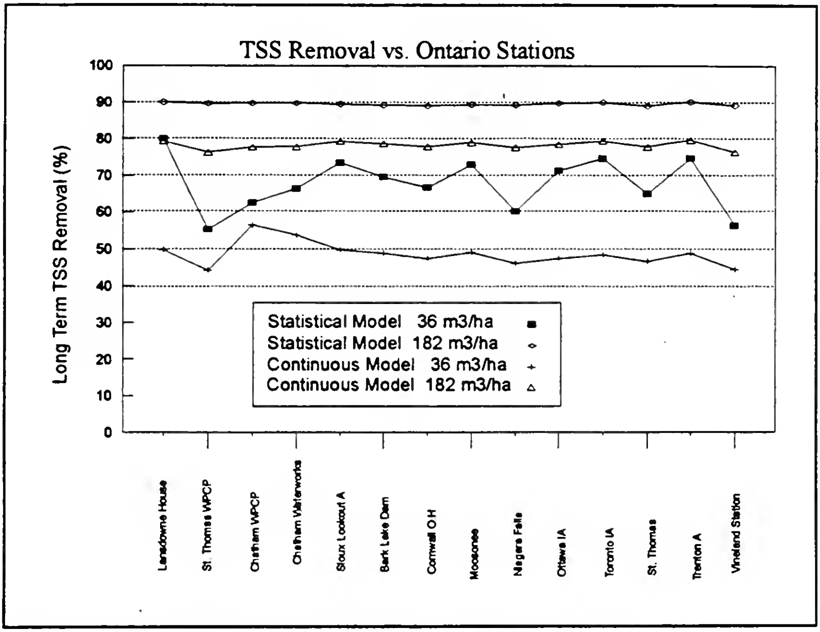


Figure 4.1 Regional Variation in TSS Removal across Ontario

Based on this finding, the SWMM and settling model were used to simulate a continuous period of 20 years using Toronto International Airport data on a one hour timestep for the various end-of-pipe stormwater management facilities, different storage volumes, and different levels of catchment imperviousness. The outlet discharge rating curves for all of the SWMPs which were simulated were based on a 24 hour drawdown of the design volume.

Three major results became apparent as a result of the modelling exercise :

1. The amount of suspended solids settling for a given design storage varies with SWMP type. (SWMPs require different volumes of storage to provide the same suspended solids removal performance)

2. The volume of water in the permanent pool of a wet facility (wet pond, wetland) is more important than the active storage component for suspended solids removal.

3. The suspended solids removal performance becomes asymptotic with increasing design storage (There is a limit to storage beyond which there are negligible increases in suspended solids settling.)

The variation in performance with SWMP type can be explained by the sedimentation model itself, the various typical configurations of the facilities, and the different removal mechanisms. For example, infiltration type SWMPs were assumed to remove 90% to 95% of the suspended solids from water which was infiltrated. This results in a high removal efficiency if the storage is large enough to contain the storm (or polluted portion of the storm). The model only looked at sedimentation, and assumed that re-suspension of previous settled pollutants would not occur. Therefore, wetlands were more effective than wet ponds since they were modelled with a shallower depth. Wet facilities were more effective than dry ponds due to the effect of the permanent pool.

The importance of the permanent pool is considerable. The simulations that were conducted indicated that a wet pond without any extended detention storage was still highly effective for solids settling. The results can be explained by the hydraulic operation of these facilities. During a storm, the influent loading is diluted in the permanent pool. Any discharge from the pond during the storm event is therefore diluted (given that the configuration of the pond is appropriately designed). After the storm has subsided there is still a considerable volume of suspended solids which is trapped in the permanent pool and has not settled. These solids have the inter-event times (ie. 2 to 3 days on average) to settle out in the pond. This combined action of dilution and inter-event settling makes wet facilities extremely efficient.

The asymptotic nature of the curves is easily explained by the frequency distribution of rainfall events. Once the storage exceeds the volume of most small runoff events, the excess storage provides no benefit.

The results from the continuous simulation modelling were used to develop curves indicating the suspended solids removal performance for various types of end-of-pipe stormwater management facilities and storage volumes. Four sets of curves were developed corresponding to the four different levels of imperviousness which were modelled.

These curves were assessed based on the recent guidelines produced by the Ministry of Natural

Resources ("Fish Habitat Protection Guidelines for Developing Areas", 1994). A subjective relationship was established between the removal curves and habitat protection criteria based on the lethal and chronic effects of suspended solids on aquatic habitat. This relationship established the required storage volumes for different types of SWMPs to protect different habitat classes as denoted in the guidelines.

It should be recognized that any water quality criteria established in the absence of subwatershed planning will have a certain degree of subjectivity. Furthermore, although fish habitat classes are used to determine the water quality criteria, they are used as an indicator of the receiving waters' sensitivity to urban development impacts. Another indicator, such as macrozoobenthic organisms could have been used. However, the water quality criteria would remain the same.

The Ministry of Natural Resources have used three levels of fish habitat in their classification system. A fourth classification has been included in this manual for the specific case of retrofit development (ie. replacing existing development with new development). Compensation should be considered in retrofit cases when the water quality criteria denoted in a level 4 protection cannot be achieved.

Descriptions of the various classes of fish habitat, and hence, water quality criteria are listed below. These water quality criteria are not intended to supersede those in the Blue Book (Water Management , Goals, Policies, Objectives and Implementation Procedures of the Ministry of Environment and Energy, 1984).

Level 1 Protection

Level 1 protection should be applied in areas with Type 1 habitat as defined in MNR's "Fish Habitat Protection Guidelines for Developing Areas" (MNR, 1994). Type 1 habitats limit the overall fisheries productive capacity. Examples of Type 1 habitat include :

- spawning areas for species with stringent spawning requirements (upwelling for brook trout; water velocity requirements for spawning walleye)
- essential rearing areas (juvenile lake trout require deep, cold, oxygenated water)
- highly productive feeding areas (wetlands)
- refuges (smallmouth bass require rocky areas for winter hiding)
- constricted migration routes
- habitats supporting endangered, threatened, or vulnerable species (as designated by the Committee on the Status of Endangered Wildlife in Canada)
- groundwater recharge areas in coldwater streams

It should be noted that the Department of Fisheries and Oceans will not accept compensation for Type 1 habitat. Therefore, development will be restricted if Level 1 protection can not be provided.

Level 2 Protection

Level 2 protection should be applied in areas with Type 2 habitat. Habitat in this classification is usually abundant and is not a limiting factor for the species productive capacity. Examples of Type 2 fish habitat include :

- feeding areas, particularly for adult fish
- areas of unspecialized spawning habitat, such as that used by many minnow species
- pool-riffle-run complexes that occur along much of a watercourse

It is expected that Level 2 protection will be applied in most cases throughout Ontario.

Level 3 Protection

Level 3 protection should be applied in areas with Type 3 fish habitat. Type 3 habitat areas have a low capacity for fish production and do not have a reasonable potential for enhancement or restoration. Examples of Type 3 habitat include :

- municipal drains
- highly altered watercourses (by urbanized or agricultural practices), or portions of waterbodies which have been hardened (concrete lined, etc.) and polluted
- artificial drainage swales

Level 4 Protection

Level 4 protection is intended only for retrofit and re-development situations. This is the minimum level of acceptable protection in these instances. It is expected that every attempt will be made to provide the appropriate level of protection as determined by the receiving water habitat in these cases. It is recognized, however, that retrofit and re-development impose physical constraints on the solutions that can be implemented. Under these circumstances Level 4 protection may be the only level of protection that is feasible.

Water quality criteria, as defined, are presented in Table 4.1. Different requirements are noted for different end-of-pipe stormwater management facilities in Table 4.1. This recognizes the difference in pollutant removal effectiveness with SWMP type. Table 4.1 also recognizes the importance of the permanent pool volume and provides required permanent pool sizing in these types of facilities. For facilities with a permanent pool (ie. wet pond, wetlands) in Table 4.1, the required storage volume is comprised of 40 m^3/ha extended detention while the remainder of the storage is permanent pool.

Table 4.1 Water Quality Storage Requirements based on Receiving Waters *					
Protection Level	SWMP Type	Storage Volume (m^3/ha) for Impervious Level			
		35 %	55 %	70 %	85 %
Level 1	Infiltration	25	30	35	40
	Wetlands	80	105	120	140
	Wet Pond	140	190	225	250
	Dry Pond (Batch)	140	190	210	235
Level 2	Infiltration	20	20	25	30
	Wetlands	60	70	80	90
	Wet Pond	90	110	130	150
	Dry Pond (Batch)	60	80	95	110
Level 3	Infiltration	20	20	20	20
	Wetlands	60	60	60	60
	Wet Pond	60	75	85	95
	Dry Pond (Batch)	40	50	55	60
	Dry Pond	90	150	200	240
Level 4	Infiltration	15	15	15	15
	Wetlands	60	60	60	60
	Wet Pond	60	60	60	65
	Dry Pond (Batch)	25	30	35	40
	Dry Pond	35	50	60	70

*** For wetlands and wet ponds all of the storage, except for 40 m^3/ha, in Table 4.1 represents the permanent pool volume. The 40 m^3/ha represents extended detention storage. Table 4.1 was based on specific design parameters (depth, length to width ratio) for each type of end-of-pipe stormwater management facility. The values of these parameters are provided in Appendix H. All values in Table 4.1 are based on a 24 hour detention.**

It should be noted that Table 4.1 only addresses water quality, and does not incorporate erosion, baseflow, and flooding concerns. These concerns, and their integration with water quality will be discussed in subsequent sections of this chapter and are discussed in detail in Chapter 3 (Sections 3.6 and 3.7).

Table 4.1 does not indicate the variation in water quality performance with storage. Therefore, Figures 4.2 through 4.5 are provided to indicate the marginal benefits or reductions in water quality performance for storage volumes which vary from those presented in Table 4.1.

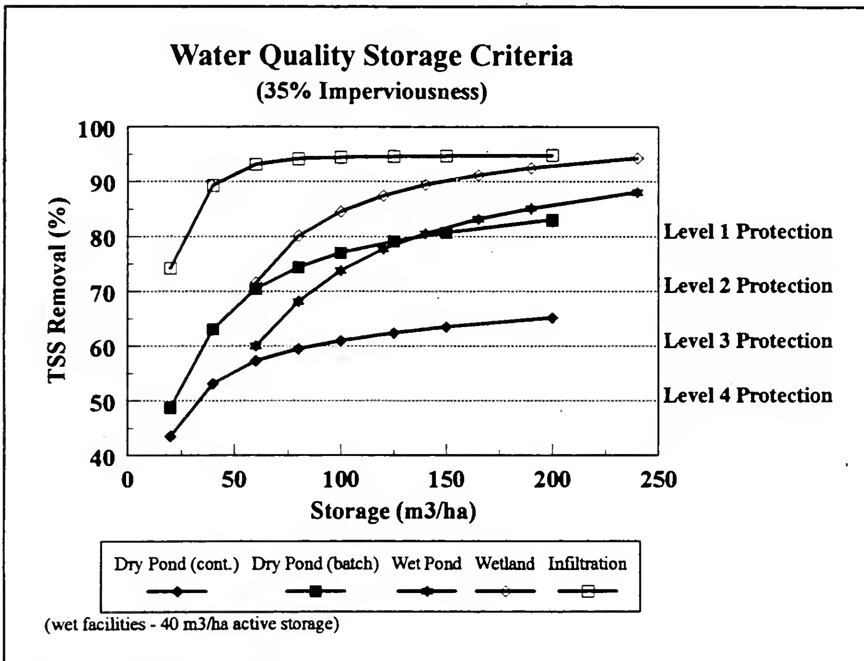


Figure 4.2 Water Quality Storage Criteria (35 % Imperviousness)

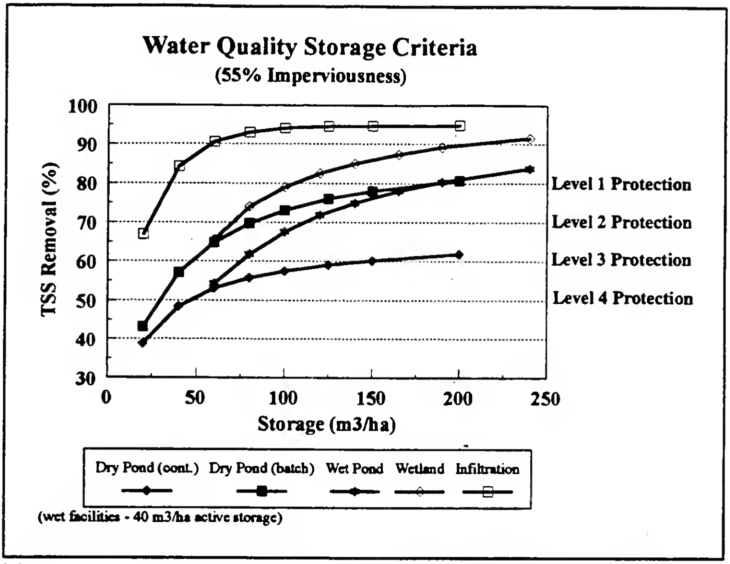


Figure 4.3 Water Quality Storage Criteria (55 % Imperviousness)

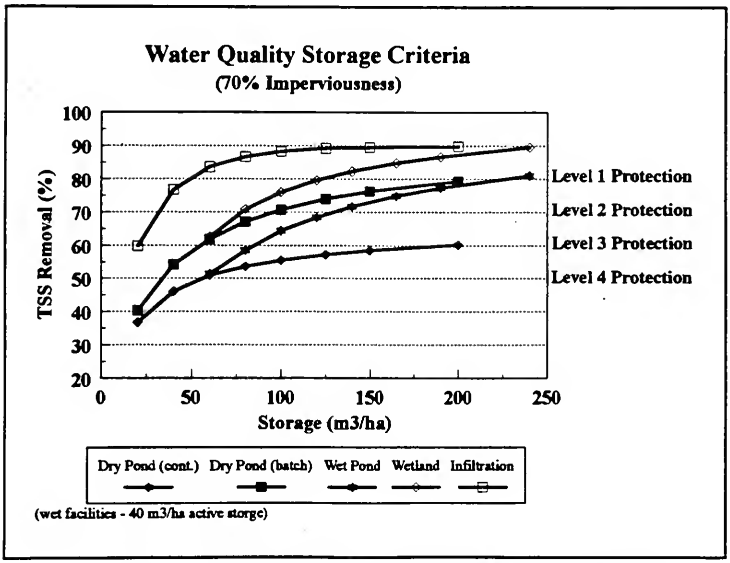


Figure 4.4 Water Quality Storage Criteria (70 % Imperviousness)

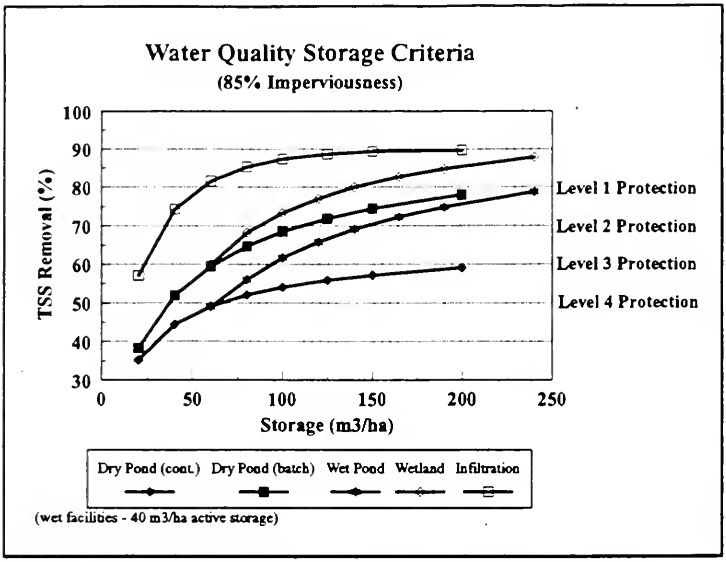


Figure 4.5 Water Quality Storage Criteria (85 % Imperviousness)

Compensation

As indicated previously, compensation will not be allowed in areas of Type 1 habitat. Compensation in this context refers to fisheries habitat compensation (ie. rehabilitation works) to comply with the Federal Fisheries Act as opposed to monetary compensation. Monetary compensation is currently being investigated by various authorities (MTRCA, CVCA) for infill and/or retrofit situations. Fisheries habitat compensation, or development restrictions, may be required for new development if the appropriate level of control cannot be provided. Monetary compensation may be required for infill and/or retrofit developments if the appropriate level of control cannot be provided.

Although compensation is not preferred, there are instances where the benefits derived from water quality controls are not commensurate with the cost of their implementation. In these situations compensation is appropriate, since the controls may be more effectively implemented in other areas of the subwatershed. It is stressed that the implementation framework for the compensation package must be established if monetary compensation is allowed. The implementation framework must ensure that monies which are levied against development proposals in lieu of on-site works must be spent on the restoration/enhancement of the subwatershed.

Recreation Concerns

Recreational concerns which involve water contact (ie. swimming) may require additional water quality controls (ultra-violet disinfection) depending on the distance between the development and the recreation area, and the contributing drainage area up to the recreation location compared to the size of development. In areas where body contact recreation is not a concern, wet stormwater management facilities and infiltration techniques adequately control bacterial loadings (faecal coliform, ecoli).

In instances where the proposed development is upstream of the recreational concern, and the development area is either greater than or equal to 10 % of the drainage area discharging to the swimming area, a subwatershed plan should be undertaken to address the cumulative impact of development on the swimming area.

Temperature Concerns

Urbanization causes temperature increases in stormwater. Ponds can compound this temperature increase since open water will tend to acclimate with the ambient air temperature. Literature values of temperature increases with different types of end-of-pipe stormwater management facilities have been recorded (Galli, 1990) and are provided in Table 4.2.

Table 4.2 Average Temperature Increases by SWMP Type*

SWMP Type	Temperature Increase
Infiltration Basin	1.4°C
Wetland (extended detention)	3.4°C**
Dry Pond (extended detention)	2.9°C
Wet Pond (extended detention)	5.1°C

* guideline based on monitoring - detailed calculations supersede this table

** original report value modified since the wetland design permanent pool was undersized resulting in a low Δt

In the absence of subwatershed planning, wet facilities (wet ponds and wetlands) may be acceptable in areas of temperature concern as long as the following conditions are met :

- stormwater lot level controls are maximized (Alternative Development Standards)
- outlet cooling is provided (subsurface discharge)
- a planting strategy and landscaping plan is prepared (both emergent and

- submergent vegetation) to shade the facility
- the facility configuration minimizes large areas of open water (minimum 4:1 length to width ratio)
- site planning techniques are investigated (preserving headwaters, minimizing topographical changes on-site, Alternative Development Standards)

A temperature mass balance can be calculated to estimate the effect of wet ponds and wetlands on temperatures in the receiving waters. The mass balance can be calculated based on Equation 4.1.

$$\Delta t = \frac{QT+q(Turb+\nabla t)}{(Q+q)} - T$$ Equation 4.1 Temperature Mass Balance

where :

Δt =	change in stream temperature (°C)
Q =	average monthly summer daily maximum flow rate in the stream (m^3/s)
T =	average monthly summer temperature in the stream (°C)
Turb =	average urban runoff summer temperature (°C)
q =	average flow from SWMP during a 15 mm storm event (m^3/s)
∇t =	average increase in temperature by SWMP type (Table 4.2)

The average urban runoff temperature (Turb) can be predicted based on a monitored relationship between urbanization and stream temperature (Galli, 1990). This relationship is shown in Equation 4.2.

$$Turb = 15.8 + 0.08\ (\%Imperviousness)$$ Equation 4.2 Urban Runoff Temperature

Table 4.2 is based on first generation end-of-pipe stormwater management facility designs and is conservative compared to current design guidelines. It is also conservative since Maryland has a warmer climate than southern Ontario. As such, further temperature monitoring of Ontario facilities is required to refine the values provided in Table 4.2.

It is suggested that the runoff from a 15 mm storm be used as the design event for the calculation of an average pond outflow rate since the 25 mm storm is too severe to estimate the effect of ponds/wetlands on daily stream temperatures. An analysis of the Toronto Bloor Street precipitation gauge indicated that the daily capture of storms up to 15 mm in size represents the capture of 80% of the annual precipitation.

The monthly average daily maximum flow rates and average monthly temperature in the stream for the month of July or August should be used in Equation 4.1. These values can be obtained from stream gauges operated by the Water Survey of Canada. The stream flow rate measured at the gauge can be areally pro-rated using Equation 4.3 to determine the value of Q at the

location of interest. In order to prevent changes to the overall thermal regime, the increase in temperature must not exceed the average summer maximum temperature for July or August in the stream.

$$Q_1 = Q_2 (A_1 / A_2)^{0.75}$$ Equation 4.3 Areal Estimation of Streamflows

where : Q_1 = flow rate at the location of interest (m^3/s)
A_1 = upstream drainage area at the location of Q_1
Q_2 = flow rate at the stream gauge (m^3/s)
A_2 = upstream drainage area at the stream gauge

Allowable temperature increases depend on the existing and/or potential temperatures in the receiving waters. Table 4.3 provides some guidance to the acceptable changes to the thermal regime in the receiving waters.

Table 4.3 Allowable Change in Thermal Regime

Receiving Water Potential	Allowable Δt
Cold Water (≤ 20°C)	1 °C
Warm Water (> 20°C)	2 °C

The restrictions imposed by these temperature guidelines can be summarized as follows :

- wet ponds are acceptable for streams with an average summer monthly temperature ≥ 24°C
- wet ponds are generally acceptable for a stream temperature of 20°C if the pond outflow < 35% of the average monthly summer daily maximum streamflow rate
- wet ponds are generally acceptable for a stream temperature of 18°C if the pond outflow < 20% of the average monthly summer daily maximum streamflow rate
- wet ponds are generally acceptable for a stream temperature of 16°C if the pond outflow < 15% of the average monthly summer daily maximum streamflow rate
- wet ponds are generally acceptable for a stream temperature of 14°C if the pond outflow < 10% of the average monthly summer daily maximum streamflow rate

This methodology does not account for the cumulative impact of numerous developments on stream temperatures in an urbanizing watershed. Subwatershed planning should be undertaken if there is significant urban growth planned for the watershed/subwatershed in the near future

(5 years) and the cumulative impact of this urbanization on stream temperatures is of concern.

4.4.2 Erosion Potential

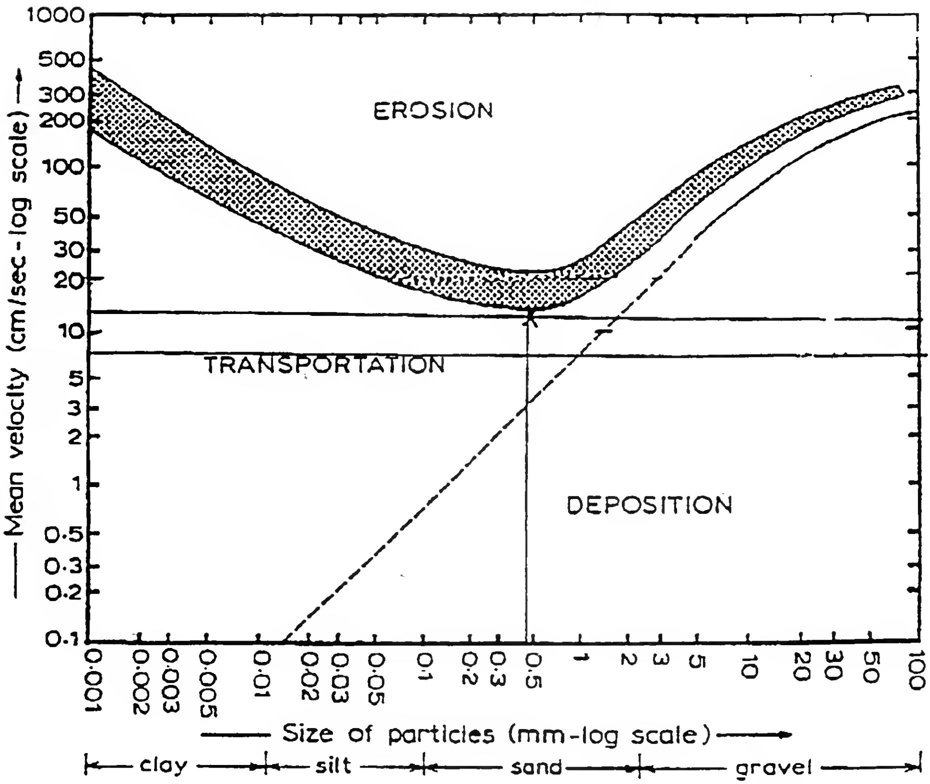
The preferred methodology to address erosion control is continuous simulation using an erosion index which is based on either tractive force or velocity-duration information. Information should be collected concerning the streambank stratigraphy in various reaches to determine the threshold for erosion in each reach of the watercourse. This type of analysis accounts for the overall change in hydrologic regime in the stream resulting from the ultimate development scenario in the subwatershed, and the variations in bank stratigraphy throughout the various reaches of the watercourse. This analysis should be conducted during a watershed/subwatershed plan since this would be excessive for single developments.

The typical criteria used to determine erosion control requirements for a single development, if a subwatershed plan has not been prepared, is to detain the runoff from a 25 mm storm (4 hour Chicago distribution) for 24 hours. The use of this storm results from an empirical observation that the stormwater quality control provided by the MOEE/MNR Interim Stormwater Quality Guidelines mitigates erosion concerns. Using this design event, the active storage is sized to detain the runoff from a 25 mm storm for 24 hours. The active storage provided by stormwater lot level and conveyance controls will reduce the end-of-pipe erosion storage requirements. Section 3.6 provides methods to calculate end-of-pipe storage benefits based on the implementation of stormwater lot level and conveyance controls.

It should be recognized however, that the use of a 25 mm storm, albeit convenient, may not provide the required erosion control since it does not take into account receiving stream conditions (banks stratigraphy, channel slope and cross-section, etc.).

The use of the 25 mm storm is an interim solution for erosion control. It is recognized that erosion is a complex natural phenomenon which, given our current level of understanding, cannot be definitively predicted (even within the context of a subwatershed plan).

A check can be made on the 25 mm control to provide a sense of whether this level of control is providing reasonable erosion control. The same tractive force/permissible velocity analysis which is recommended for implementation at a subwatershed plan level can be applied for the 25 mm storm event simulation. An assessment of the immediate downstream channel can be represented by a mean particle size diameter that will be subject to tractive forces. This particle size can be converted into a mean velocity, above which, erosion will occur. A simple chart such as the one provided in Figure 4.6 (Hjulstrom, 1939) can be used to estimate this permissible velocity. Based on the existing downstream channel configuration (cross-section, slope, ie. stage-storage-discharge relationship) an erosion index for the pre-development hydrograph during the 25 mm storm event can be compared to that under post-development situations. The erosion index calculation is provided in Equation 4.4.



SOURCE :'GEOMORPHOLOGY IN ENVIRONMENTAL MANAGEMENT'
R.U. COOKE, J.C. DOORNKAMP, CLARENDON PRESS-OXFORD, 1978

Figure 4.6 Erosion Potential Based on Soil Particle Sizes

$$E_i = \sum (V_t - V_c)\, \Delta t$$ Equation 4.4 Erosion Index

where E_i = Erosion Index
V_t = Velocity in the channel at time t ($> V_c$) (m/s)
V_c = Critical velocity above which erosion will occur (m/s)
Δt = timestep (s)

The summation on the right side of Equation 4.4 is performed for the entire duration of the storm during the period of time when the discharge velocity is greater than the critical velocity. Several things must be remembered when using this check :

- a flow must be assumed in the receiving waters (either pro-rated based on existing flow data - Master Drainage Plans, flow gauge data) or calculated based on a single lumped basin approach) during the 25 mm storm as a starting receiving water condition
- the flow from the subdivision/site after modulation by the SWMP(s) should be used in the post development analysis

Further research is required to define a better linkage between receiving water conditions and erosion control. The establishment of a relationship between erosion control requirements and stream classification (ie. Rosgen classifications) would be an improvement over the current interim requirements, and should therefore be investigated.

Rehabilitation of existing erosion areas is best identified as part of the subwatershed plan. In these cases a subwatershed plan may trigger a more detailed study which deals with the rehabilitation of the identified problem areas within the subwatershed. Piecemeal fixes to erosion are generally discouraged since the ultimate development scenario and flow regime is unknown without subwatershed planning. Any rehabilitative works that are proposed should employ natural channel design techniques where possible. Guidance on natural channel design techniques is provided in "Natural Channel Systems : An Approach to Management and Design, Development Draft" (Ministry of Natural Resources, 1994).

The derived extended detention storage for erosion control should be compared to the extended detention storage criteria in Table 4.1. For infiltration and dry pond type SWMPs in Table 4.1, all of the storage volume represents extended detention storage. For wet type facilities (wetlands, wet ponds) only 40 m^3/ha of the indicated storage volume represents extended detention.

The larger of the two extended detention storage values (erosion or water quality active storage) should be used in the design of the SWMP. Permanent pool sizing for wet ponds and wetlands should still be based on Table 4.1 (indicated storage volume minus 40 m^3/ha) according to the appropriate level of stream protection.

4.4.3 Water Quantity

Post to pre-development peak flow control may result in higher flows instream when applied across a watershed, due to volume increases and timing effects. Therefore, flood control requirements should be determined on a watershed/subwatershed basis through an assessment of potential flood hazards (eg. bridge crossings, flood vulnerable areas, active valley land uses, etc.)

In the absence of subwatershed planning, the location of the downstream potential flood hazards will dictate the requirements for water quantity control. If a potential flood hazard area is located immediately downstream of the proposed site, water quantity control must be implemented.

Although an assessment of cumulative impacts cannot be made without the preparation of a subwatershed plan, certain hydrologic/hydraulic principles should be reviewed before a criteria is established for water quantity control. Water quantity control, from a watershed/subwatershed perspective, is likely to be most effective if implemented in the headwaters of the watershed/subwatershed, and least effective in the lower reaches of the watershed/subwatershed. This effect occurs because of the hydrograph elongation that occurs with extended detention storage.

Extended detention does not affect the volume of runoff, but merely reduces the peak flow rate. The lack of volume treatment results in a much longer period of time where there are "near" peak flows. In the situation where extended detention is utilized in a downstream reach, water from the upstream areas can "catch up" with the near peak flows being released from the lower areas resulting in an overall increase in peak flow. If extended detention is utilized in the headwaters the coincidence of peaks is less likely to occur.

Therefore the following generic recommendations are suggested for consideration in determining the water quantity criteria in the absence of subwatershed planning :

- if there is a potential flood hazard immediately downstream of the proposed site, water quantity control must be implemented.
- if the development is located in the headwater areas, the post development peak flow rates should be controlled to pre-development levels
- if the development is located in the lower reaches of the watershed either no quantity control would be required, or over-control would be required (if there were potential flooding concerns immediately downstream of the proposed development)

Although these generic principles should be considered, <u>the site specific characteristics</u> of the site should dictate the appropriate quantity control measures.

Typical water quantity requirements include controlling post development peak flows to pre-development levels for the 2, 5, 25, and 100 year storms. Some municipalities require the modelling of the 10 year storm in addition to these storms since their storm sewer system is designed to accommodate this storm. In cases where over-control is requested, the 2 and 5 year post-development peak flows are typically controlled to 50% of their pre-development levels.

The typical storm distribution which is used in Ontario for water quantity control is the 4 hour Chicago distribution. This distribution uses local intensity duration frequency information to derive a rainfall hyetograph. Although this is the typical standard, other storm distributions are usually acceptable depending on the land use and size of area being modelled (3 hour Chicago, 6, 12, or 24 hour SCS, 1 or 12 hour AES).

Some agencies require that several storm distributions be screened, and that the distribution which produces the highest uncontrolled peak flows be used. The Chicago distribution is intense and is representative for small urban areas (< 100 ha). The longer distributions (6 hour, 12 hour, 24 hour) are generally representative of larger areas, and/or areas with low imperviousness.

The timestep which is chosen for modelling should be representative of the time of concentration. Choosing a timestep which is too large can "miss the peak" whereby a timestep falls on either side of peak runoff flowrate. In addition, Chicago distributions with very small timesteps (< 10 min) can result in unrealistically intense hyetographs. In these cases, real storms should be reviewed to ensure that the hyetograph is realistic. Smaller timestep hyetographs can be created from longer (10 minute) hyetographs by duplicating the longer timestep intensities.

The Regional storm (Hurricane Hazel in southern Ontario, Timmins storm in northern Ontario) is generally modelled for large areas (> 100 ha) but not for smaller areas, since the 100 year storm produces higher flows than the Regional storm for small catchment areas. The regulatory storm is chosen as either the Regional storm or the 100 year storm, depending on which one produces the highest peak flows for the area in question. The regulatory storm is generally not controlled, but is assessed with respect to the adequacy of the overland flow conveyance system.

The event modelling should recognize and incorporate the storage being provided by the stormwater lot level controls, conveyance controls, and end-of-pipe stormwater management quality/erosion facilities. Chapter 3 (Section 3.6) provides guidance for the assessment of storage benefits provided by stormwater lot level, conveyance, and end-of-pipe SWMPs with respect to water quantity control.

4.4.4 Baseflow Maintenance

The methodology for assessing stormwater management measures ensures that the groundwater

contribution to baseflow is considered (see Section 4.5). No runoff from a 5 mm storm should occur for any development (excluding roads) in these areas as a minimum level of control for baseflow maintenance.

If the areas in question discharge to a first order stream, and/or there is a downstream area with a Type 1 habitat (spawning areas for species with stringent spawning requirements, essential rearing areas, highly productive feeding areas, refuges, constricted migration routes, and habitats supporting endangered species), stringent development controls/restrictions will be required to maintain baseflow in the stream in the absence of subwatershed planning. These restrictions include but are not limited to :

- development forms which preserve headwater discharge areas
- development forms which protect groundwater recharge areas
- reducing lot grades (ie. 2% to 0.5%) to promote depression storage and subsequent evapotranspiration/infiltration (Making Choices : Alternative Development Standards Guidelines, Ministry of Housing and Ministry of Municipal Affairs, 1994)
- ditch and culvert servicing, or pervious pipe servicing with anti-seepage collars, or regular servicing with pervious catch-basins
- preserving the existing topography in the development layout
- use of foundation drain sump pumps to either the surface or soakaway pits

Continuous modelling of low flows for small areas is generally discouraged. In instances where continuous modelling can be calibrated with at least 1 year's worth of flow monitoring, a comparison can be made of the 2 year 3 day low flow values (3Q2) under existing and developed conditions. The 3Q2 under developed conditions should equal that under pre-development conditions.

4.5 SWMP Selection

The goal of stormwater management, whether a Subwatershed Plan has been prepared or not, is to preserve the natural hydrologic cycle. The preparation of a subwatershed plan, from a water management perspective, will outline the volume of control required to ensure that there are no cumulative impacts with respect to downstream flooding, water quality degradation, erosion problems, baseflows, and other concerns (nutrients, temperature, dissolved oxygen, bacteria). If a subwatershed plan has not been prepared, **there is no way that an individual development plan can explicitly address cumulative impacts**.

The preservation of the natural hydrologic cycle, however, minimizes the potential for cumulative downstream flooding, erosion, and baseflow impacts. Accordingly, the methodology for the assessment of stormwater management techniques does not change whether a subwatershed plan has been completed or not.

The preferred methodology involves assessing a hierarchy of stormwater management practices. Generally a combination of techniques will be required to address the overall set of water management concerns. Stormwater management measures should be assessed in the following order :

1. Stormwater lot level controls
2. Stormwater conveyance controls
3. End-of-pipe stormwater management facilities

<u>Stormwater Lot Level Controls</u>

In all instances of development, stormwater lot level controls should be investigated first. Stormwater lot level controls include but are not limited to the following practices :

- rain leader discharge to surface infiltration areas in rear yards
- rain leader discharge to subsurface soakaway pits with an overflow discharge to the surface
- reducing lot grading from 2% to a minimum of 0.5%
- separate foundation drains or
    - foundation drain sump pumps to surface discharge
    - foundation drain sump pumps to soakaway pits

It should be noted that there may be by-laws restricting stormwater lot level controls in some municipal jurisdictions. Municipal by-laws should be investigated to ensure lot level controls are legally as well as technically feasible.

Examples of stormwater lot level controls and technical information concerning the design of these SWMPs are provided in detail in Chapter 3 (Section 3.2).

<u>Stormwater Conveyance Controls</u>

Stormwater conveyance controls refer to perforated pipe systems (storm sewers), pervious catch-basins, and grassed swales.

Many agencies recognize the benefits of ditch and culvert servicing and grassed swales. These types of stormwater conveyance controls are generally promoted. Municipal by-laws must be scrutinized, however, to ensure that ditch and culvert servicing is an acceptable form of servicing with the subject municipality.

Several municipalities have implemented pervious storm sewer systems (Etobicoke, Manotick, Nepean, Tokyo) and/or pervious catch-basins. In many cases of infrastructure renewal, pervious

pipes and pervious catch-basins will be the only options available due to land constraints. There is considerably more controversy with respect to these types of stormwater conveyance controls since contaminated runoff from the roads is infiltrated. As such, these types of SWMPs may be subject to the MOEE Reasonable Use Policy and groundwater studies may be required to demonstrate that groundwater will not become contaminated. The application of Reasonable Use is usually site specific and dependent on whether there is the potential for exfiltrated stormwater to contaminate a deep groundwater aquifer system, or a shallow system if it is being used for drinking water. If there is reasonable certainty that the exfiltrated water will only enter a shallow groundwater system which discharges to a nearby surface stream, Reasonable Use is generally not being applied by the MOEE Regional staff.

The use of these types of SWMPs will depend, to a great extent, on the municipality's willingness to implement and maintain them. Efforts should be made during the formulation of a SWMP strategy to determine whether stormwater conveyance controls are acceptable stormwater management options.

Examples of stormwater conveyance controls and technical information concerning the design of these measures are provided in detail in Chapter 3 (Section 3.3).

End-of-Pipe Stormwater Management Facilities

End-of-pipe stormwater management facilities receive stormwater from a conveyance system (ditches, sewers) and discharge the treated water to the receiving waters. There are nine generalized categories of end-of-pipe stormwater management facilities :

- wet ponds
- wetlands
- dry ponds
- infiltration basins
- infiltration trenches
- filter strips
- buffer strips
- sand filters
- oil/grit separators

Illustrations of the various end-of-pipe stormwater management facilities and technical design information for these measures are provided in Chapter 3 (Section 3.4).

The selection and design of end-of-pipe stormwater management facilities in the absence of subwatershed planning is driven by the receiving water concerns that have been identified. The receiving water concerns are reflected by the subdivision/site planning, and water quality, erosion, and quantity control criteria that have been determined (baseflow maintenance is addressed by the hierarchical method used to assess stormwater management measures).

Given that there is a preferred hierarchy of stormwater management measures, and that the receiving water concerns are reflected in the criteria, the selection of appropriate water management measures is based on four factors :

- Physical suitability
- Conformity with development plan
- Cost
- Technical longevity/effectiveness

The end-of-pipe stormwater management facility selection process in general is inclusive. There is a general consensus that the type of facility and level of control should be dictated by the sensitivity of the receiving water concern. There is no receiving water condition which precludes the use of a SWMP, however, since a second SWMP can always be added to mitigate the negative impacts of the first SWMP. That is, receiving water concerns do not rule out the use of SWMPs, but may require the use of multiple SWMPs to ensure that all concerns are dealt with.

For example, a temperature concern can be mitigated by a wet pond with : an appropriate pond configuration and planting strategy, riparian stream cover planting strategy, aquatic habitat restoration, outlet cooling, or any combination of these techniques. To say that there is a sensitivity which rules out the use of wet ponds is inappropriate unless it can be demonstrated through an impact assessment (continuous simulation) at the subwatershed plan level.

4.5.1 Physical Suitability

Physical factors relate to the individual physical characteristics of the proposed development site. Factors which need to be assessed in terms of the stormwater management measures being physically feasible include :

- topography
- soils stratification
- depth to bedrock
- depth to seasonally high water table
- drainage area

Detailed information on the physical constraints which would limit the implementation of stormwater lot level, conveyance, and end-of-pipe stormwater management facilities is provided in Chapter 3. Table 4.4 is provided as a summary of this information, and indicates the physical conditions required for a SWMP to be physically feasible. **Physical feasibility is the only constraint that will exclude a SWMP from further consideration during the selection process.**

Table 4.4 Physical Criteria for SWMP Types					
SWMP	Topography	Soils	Bedrock	Groundwater	Area
wet pond	none	none	none	none	> 5 ha
dry pond	none	none	none	none	> 5 ha
wetland	none	none	none	none	> 5 ha
infiltration basin	none	loam (min. inf. rate ≥ 15 mm/h)	> 1 m below bottom	> 1 m below bottom	< 5 ha
infiltration trench	none	loam (min. inf. rate ≥ 15 mm/h)	> 1 m below bottom	> 1 m below bottom	< 2 ha
flat lot grading	< 5%	none	none	none	none
soakaway pit	none	loam (min. inf. rate ≥ 15 mm/h)	> 1 m below bottom	> 1 m below bottom	< 0.5 ha
rear yard infiltration	< 2%	loam (min. inf. rate ≥ 15 mm/h)	> 1 m below bottom	> 1 m below bottom	< 0.5 ha
grassed swales	< 5%	none	none	none	none
perforated pipes	none	loam (min. inf. rate ≥ 15 mm/h)	> 1 m below bottom	> 1 m below bottom	none
pervious catch-basins	none	loam (min. inf. rate ≥ 15 mm/h)	> 1 m below bottom	> 1 m below bottom	none
filter strips	< 10 %	none	none	> 0.5 m below bottom	< 2 ha
sand filters	none	none	none	> 0.5 m below bottom	< 5 ha
oil/grit separators	none	none	none	none	< 1 ha

4.5.2 Conformity with the Development Plan

An appropriate stormwater management plan cannot be implemented unless there is sufficient area available for the implementation of stormwater management measures. Although this may seem obvious, many of the current stormwater management problems revolve around a lack of area available for stormwater management as a result of poor subdivision/site planning.

For example, stormwater lot level controls for roof drainage cannot be implemented for extremely compact build forms unless there are communal greenspace areas established for soakaway pits or surface ponding.

This problem exemplifies the need for subdivision/site planning which integrates the stormwater management needs with the development layout.

4.5.3 Cost

The cost-effectiveness of different combinations of SWMPs must be considered when selecting a SWMP plan. In most cases, generic costing information can be used to rank SWMP solutions in this regard. This type of planning level costing is provided in Chapter 6. It should be recognized, however, that the costs presented in Chapter 6 do not account for site specific construction difficulties, and therefore should not be used in the preparation of detailed cost projections.

The most cost-effective stormwater management measures that meet the overall receiving water objectives/criteria should be implemented.

4.5.4 Technical Effectiveness/Longevity

There is approximately a decade of widespread experience with the implementation of SWMPs in North America. This experience has revealed that some SWMPs are more susceptible to clogging and/or less effective at removing pollutants than others.

This information can be used in the selection of SWMP solutions. A technical effectiveness/performance ranking can be assigned for each type of SWMP based on Table 4.5. In areas where more than one type of SWMP is proposed, an overall technical effectiveness can be calculated based on an areal weighting of individual SWMPs and the drainage area which they service.

Any SWMPs with a technical effectiveness rating of 5 or less that are proposed must include pre-treatment options, a stringent maintenance plan, and have specific maintenance requirements included in the Certificate of Approval.

Any SWMPs with a technical effectiveness rating of 3 or less must include either :

a) new innovation in terms of design features to increase performance and longevity compared to the current standard design
   or
b) a contingency plan to retrofit or replace the SWMP and a proposed compliance monitoring program

For example, an infiltration basin design would be acceptable if a contingency plan to retrofit the basin into a wet pond was submitted for review along with a proposed compliance monitoring program.

Table 4.5 Technical Effectiveness/Longevity of Different SWMPs*	
Extended Detention Wet Ponds	10
Extended Detention Constructed Wetlands	9
Sand Filters	8
Rear yard infiltration (non-standard grading)	7
Grassed Swales	7
Extended Detention Dry Ponds	7
Roof Leader Soakaway Pits	6
Filter Strip	5
Pervious pipe storm sewers	4
Infiltration Trenches	4
Dry weather/Manhole oil/grit separator	4
Pervious bottom catch-basins	3
Infiltration Basins	2
3 chamber oil/grit separator	2

* Rating is out of 10 (10 signifies excellent performance, 1 signifies poor performance)

The ratings provided in Table 4.5 are provided as guidance and represent an opinion based on past experience. For a particular site these ratings may be changed based on the experience of the stormwater management professionals involved in the design and review of water management measures, and their knowledge of the site conditions. An agreed-upon rating of SWMP technical effectiveness/longevity should be established by the stormwater management plan designer in concert with the reviewing agencies at the outset of the development planning.

5.0 OPERATIONS AND MAINTENANCE

5.1 History of Stormwater Management O & M

The operations and maintenance of stormwater management measures is currently a contentious issue. During the 1970s and 1980s stormwater management consisted of "peak shaving" facilities whereby the peak flow under post development conditions was reduced to that under pre-development conditions (2 year control for erosion, and up to 100 year for flooding). These facilities did not require sediment removal maintenance since the residence time of water within a "peak shaving" facility was in the order of several hours and there was marginal pollutant removal. Since these facilities only detained water for a short period of time, they did not restrict the active use of the land and could be combined with active parkland. For example, several baseball diamonds have been constructed in stormwater quantity facilities across Ontario. This practice continues today where separate facilities are provided for water quantity and water quality. In these circumstances, as with all peak shaving facilities, stormwater management measures were designed to require as little maintenance as possible (It should be noted that regular maintenance of quantity control facilities is still required, ie. inlet/outlet inspections, emergency spillway repair after a flood, trash removal, etc.).

The introduction of stormwater management measures to enhance water quality has changed the need for operations and maintenance. By definition, urban water quality SWMPs require sediment removal maintenance since they are designed to remove pollutants. Many of these pollutants bind to sediment (nutrients, metals, bacteria), and as such, the design of urban water quality SWMPs is currently based primarily on sedimentation. It is anticipated that other design parameters (nutrient loading, temperature) will be used in the future as our understanding of the physical, biological, and chemical pathways of these pollutants increases.

Attitudes towards stormwater management maintenance are currently in a state of flux. It has been assumed that local or regional municipalities would be responsible for the maintenance of urban water quality SWMPs since they have the work force, the equipment, and are currently responsible for stormwater infrastructure. Most municipalities are concerned with the maintenance and the possible liability associated with accumulated pollutants in a stormwater management facility. Therefore, municipalities must take part in the review and approval of facilities and ensure that equipment is available and maintenance procedures are in place to adequately operate and maintain them.

Some municipalities are reluctant to re-introduce ditch and culvert servicing instead of curb and gutter since there are additional perceived maintenance responsibilities associated with ditches and culverts. This maintenance, however, is a result of achieving the objectives of implementing ditch and culvert servicing (ie. pollutant/trash removal). Accordingly, attitudes towards SWMP maintenance must change if environmental objectives are to be achieved.

This chapter focuses on the operational and maintenance requirements for urban SWMPs which are implemented to address urban stormwater. Maintenance and monitoring is also required for watershed/subwatershed projects such as natural channel designs and canopy cover restoration. The monitoring and maintenance responsibilities for these projects should be clearly defined in the implementation strategy for the watershed/subwatershed plan and is not addressed in this document.

5.2 The Need for Maintenance

Maintenance is a necessary and important aspect of urban SWMP design. One of the main reasons for historical SWMP failures and/or poor performance is a lack of maintenance. Most urban SWMPs are not currently maintained since this is perceived to be an additional cost (in actuality, it is a cost which we have been deferring until the present).

Urban SWMP designers should give considerable thought during the design of stormwater management practices with respect to how future maintenance will be accomplished. Measures to facilitate maintenance should be implemented wherever possible considering the long term responsibility of the municipality in this regard.

Where necessary, Certificates of Approval (C of A) for water quality facilities should clearly define maintenance responsibilities (inspection to determine when maintenance is required, maintenance practices to be performed, reporting procedures for maintenance). Future Certificates of Approval may contain conditions related to the operation and maintenance of the facility (eg. an annual maintenance report).

One annual maintenance report should be completed by the municipality which covers all of the water quality/recharge SWMPs in the municipality's jurisdiction. For large cities several reports could be produced for various geographic locations within the municipality. The report should provide a summary of the following items :

- observations resulting from the inspection of the SWMP over the course of the year. These observations should include comments on the :
    - hydraulic operation of the facility (detention time, evidence or occurrence of overflows)
    - condition of vegetation in and around facility
    - occurrence of obstructions at the inlet and outlet
    - evidence of spills and oil/grease contamination
    - frequency of trash build-up
- measured sediment depths in the SWMP (non-infiltration type SWMPs)
- monitoring results, if flow or quality monitoring was undertaken during the year
- maintenance and operational control undertaken for the SWMP during the year
- recommendations for the SWMP inspection and maintenance program for the coming year

5.3 Operations and Maintenance Activities

The operations and maintenance activities vary with the type of SWMP that is implemented. Table 5.1 provides a list of required operations and maintenance activities for different types of SWMPs.

5.4 Maintenance Tasks and Frequency

One of the most frequent questions concerns the frequency of maintenance. The required frequency of maintenance for most activities is not definitively known since very few facilities have been monitored to determine when maintenance should occur. Most of the monitoring which has been undertaken focuses on the pollutant removal efficiency of these SWMPs and not the maintenance/operations aspect of the facility.

Without a solid base of information on which to base maintenance frequency, many of the maintenance tasks will be performed on an "as required" basis. Theoretical estimations for the frequency of sediment removal are made for end-of-pipe stormwater management facilities in Section 5.5. These estimations are based on numerous assumptions which may not hold true for individual developments. For example, if a facility is subject to stormwater from an upstream area undergoing development, there is a greater potential for increased maintenance frequency (depending on the effectiveness of the sediment and erosion control techniques employed during the construction phase of the upstream development). There are many factors that relate to the frequency of maintenance and operational control (land use, upstream development, wildlife, etc.).

The following sections provide some guidance on how to determine when maintenance is required, and how to perform the maintenance once it is determined that maintenance is required.

Table 5.1 Stormwater Management Practices Operation and Maintenance Activities

Item No.	Operation or Maintenance Activity	Type of Stormwater Management Practice												
		Wet Pond	Wetland	Dry Pond	Infiltration Basin	Infiltration Trench	Filter Strip	Buffer Strip	Sand Filter	Oil/Grit Separator	Soak-away Pit	Pervious Pipe	Perviou s Catch-basin	Grasse d Swales
1	Inspection	■	■	■	■	■	■	■	■	■	■	■	■	■
2	Grass cutting	■	■	■	■	■			■					■
3	Weed Control	■	■	■	■		■							■
4	Upland vegetation replanting	■	■	■	■	■	■							
5	Shoreline Fringe and Flood Fringe vegetation replanting	■	■											
6	Aquatic vegetation replanting	■	■											
7	Removal of accumulated sediments	■	■	■	■	■	■		■	■		■**	■**	■
8	Outlet valve adjustment	■	■	■										
9	Roof leader filter cleaning/replacement										■			
10	Pervious pipe flushing											■		
11	Oil/grit separator or Catch-basin cleaning									■		■	■	
12	Closing of infiltration facility inlet for winter months				■●	■●			■●			■●	■●	
13	Trash removal	■	■	■	■	■	■	■	■	■*	■***	■	■*	■
14	Infiltration basin floor tilling				■									

* litter removal part of sediment removal ** sediment removal part of catch-basin cleaning *** litter removal by a filter in the rain gutter

● based on municipality experience and practices (eg. may not be required if used on a local road with no salting or sanding)

5.4.1 Inspections

Inspections of the SWMP will indicate whether maintenance is required or not. Inspections should be made of SWMPs after every significant storm during the first two years of operation to ensure that the SWMP is functioning properly. It is anticipated that this will translate into an average of four inspections per year.

After this initial period, when the SWMP operation has been confirmed, annual inspections may suffice. A greater number of inspections may be required if the SWMP is poorly designed, or if there are other factors (eg. upstream development) which cause operational or maintenance problems.

As shown in Table 5.1, regular inspections are required for all SWMP types. Table 5.2 lists the points that should be checked when inspecting various SWMPs.

5.4.2 Grass Cutting

Frequency

Grass cutting is one maintenance activity which is solely undertaken to enhance the aesthetics, or "perceived aesthetics" of the facility. Generally, it is recommended that grass cutting be eliminated around SWMPs. No grass cutting will enhance water quality and provide additional safety benefits for wet facilities. The frequency of grass cutting, however, will depend on the surrounding land uses, and local municipal by-laws. If the SWMP is located adjacent to homeowners, they may demand a manicured appearance. As such, grass cutting should be done as infrequently as possible, recognizing the aesthetic concerns of nearby residents.

Methods

Grass cutting is fairly straight forward for most SWMPs. Grass cutting around wet facilities should ensure that the grass is not cut to the edge of the permanent pool. In addition grass cutting should be done parallel to the shoreline (as a safety precaution) with grass clippings being ejected upland (to reduce the potential for organic loadings to the pond).

Table 5.2 Points of Regular Inspection

SWMP	Inspection Routine
Wet Ponds Wetlands	1. Is the pond level higher than the normal permanent pool elevation > 24 hours after a storm ? (This could indicate blockage of the outlet by trash or sediment. Visually inspect the outlet structure for debris or blockage.) 2. Is the pond level lower than the normal permanent pool elevation ? (This could indicate a blockage of the inlet. Visually inspect the inlet structure for debris or blockage.) 3. Is the vegetation around the pond dead ? Is the pond all open water (no bulrushes or vegetation in the water) ? Are there areas around the pond with easy access to open water ? (This will indicate a need to re-vegetate the pond) 4. Is there an oily sheen on the water near the inlet or outlet ? Is the water frothy ? Is there an unusual colouring to the water ? (This will indicate the occurrence of an oil or industrial spill and the need for cleanup). 5. Check the sediment depth in pond. (This will indicate the need for sediment removal. The sediment depth can be checked using a gradated pole with a flat plate attached to the bottom. A marker (pole, buoy) should be place in the pond to indicate the spot(s) where a measurement should be made. A visual inspection on the pond depth can also be made if the pond is shallow and a gradated marker is located in the pond.)
Dry Ponds	1. Is there standing water in the pond > 24 hours after a storm ? (This could indicate blockage of the outlet by trash or sediment. Visually inspect the outlet structure for debris or blockage.) 2. Is the pond always dry, or relatively dry within 24 hours of a storm ? (This could indicate a blockage of the inlet or too large of a water quality/erosion control outlet. Visually inspect the inlet structure for debris or blockage.) 3. Is the vegetation around the pond dead ? Are there areas around the pond with easy access to open water ? (This will indicate a need to re-vegetate the pond) 4. Is there a visible accumulation of sediment in the bottom of the pond or around the high water line of the pond ? (This will indicate the need for sediment removal.)

Table 5.2 Points of Regular Inspection (continued)

Infiltration Basins	1. Is there standing water in the basin > 24 hours after a storm ? (This will indicate a decrease in the permeability of the underlying soils and, depending on the depth of water in the pond after 24 hours, the need for maintenance - sediment removal and roto-tilling of soils. If there is greater than one third the design depth of water in the pond 48 hours after a storm, the basin needs to be maintained.) 2. Is the pond always dry, or relatively dry within 24 hours of a storm ? (This could indicate a blockage of the inlet. Visually inspect the inlet structure for debris or blockage.) 3. Is there a visible accumulation of sediment in the bottom of the pond or around the high water line of the pond ? (This will indicate the need for sediment removal.)
Infiltration Trenches	1. Is the trench draining ? (Inspect the depth of water in the observation well. If the trench has not drained in 24 hours, the inlet and pre-treatment SWMPs should be cleaned (ie oil/grit separator, catch-basins, or grassed swales). If the trench has not drained within 48 hours the trench may need to be partially or wholly re-constructed to maintain its performance. 2. Is the trench always dry, or relatively dry within 24 hours of a storm ? (This could indicate a blockage of the inlet. Visually inspect the inlet structure for debris or blockage.)
Filter Strips	1. Are there areas of dead or no vegetation downstream of the level spreader ? (This will indicate the need to re-vegetate the filter strip.) 2. Are there indications or rill erosion downstream of the level spreader ? (This will indicate the need to re-vegetate the filter strip. The rill erosion may be caused by a non-uniform spreader height. The spreader should be checked near the erosion areas to determine if it is need of repair.) 3. Is there erosion of the level spreader ? (The spreader should be re-constructed in areas where the spreader height is non uniform) 4. Is there standing water upstream of the level spreader ? (This will indicate that the level spreader is blocked. The level spreader should be checked for trash, debris, or sedimentation. The blockage should be removed and the spreader re-constructed if necessary).

Table 5.2 Points of Regular Inspection (continued)

Buffer Strips	1.	Are there areas of dead vegetation along the buffer strip ? (This will indicate the need to re-vegetate the buffer strip).
Sand Filters	1.	Are there areas of dead vegetation in a grass surfaced sand filter ? (This will indicate the need to re-vegetate the filter surface).
	2.	Is there standing water in the filter > 24 hours after a storm ? (This will indicate a blockage in the filter, possibly in the perforated pipe collection system or sedimentation on the surface or in the sand layer. The outlet collection system should be inspected for blockage. If there is water in the filter 48 hours after a storm, sediment removal should be undertaken. If sediment removal does not improve the performance (drainage) of the filter, the filter may need to be re-constructed).
	3.	Is the filter always dry ? (This could indicate a blockage of the inlet. Visually inspect the inlet structure for debris or blockage.)
	4.	Is there a visible accumulation of sediment in a grassed covered sand filter ? (This will indicate the need for sediment removal.)
Oil/Grit Separators	1.	Is there sediment in the separator/ catch-basin ? (The level of sediment should be measured using a gradated pole with a flat plate attached to the bottom. The pole should be gradated such that the true bottom of the separator, catch-basin compared to the cover/grate is marked for comparison.)
	2.	Is there oil in the separator/ catch-basin ? (A visual inspection of the contents should be made from the surface for trash/debris and/or the presence of a oil/industrial spill. An oily sheen, frothing or unusual colouring to the water will indicate the occurrence of an oil or industrial spill. The separator / catch-basin should be cleaned in the event of spill contamination.)
Roof Leader Discharge to Soakaway Pits	1.	Are there frequent overflows to the surface during small storm events ? (Frequent overflows will indicate that roof leader filter has clogged or the soakaway storage media has become clogged. The filter should be checked for an accumulation of leaves and twigs. If the filter is clean, the pit may need to be reconstructed to maintain its performance.)

Table 5.2 Points of Regular Inspection (continued)	
Perforated Pipe Systems	1. Are the pre-treatment SWMPs operating properly ? (Pre-treatment SWMPs should be inspected (see oil/grit separators, grassed swales).) 2. Is the perforated pipe operating properly ? (The connection to the perforated pipe (ie. manhole/catch-basin) should be visually inspected for standing water 24 hours after a storm. Standing water will indicate the need for maintenance of the perforated pipe system (flushing, jet washing)).
Pervious Catch-basins	1. Is sediment collecting in the catch-basin ? (The level of sediment in the catch-basin sump should be measured using a gradated pole with a flat plate attached to the bottom. The pole should be gradated such that the true bottom of the catch-basin compared to the grate is marked for comparison.) 2. Is there oil in the catch-basin ? (A visual inspection of the catch-basin contents should be made from the surface for trash/debris and/or the presence of a oil/industrial spill. An oily sheen, frothing or unusual colouring to the water will indicate the occurrence of an oil or industrial spill. The separator / catch-basin should be cleaned in the event of spill contamination.) 3. Is the catch-basin infiltration system operating properly ? The catch-basin should be visually inspected for standing water near the invert of the regular storm sewer 24 hours after a storm.
Grassed Swales	1. Is there standing water in an enhanced grass swale. (This will indicate a blocked check dam or decrease in the permeability of the swale. The check dam should be inspected for blockage by trash/debris, or sediment.) 2. Is the grass/vegetation dead ? (This will indicate the need to revegetate the swale). 3. Is there erosion downstream of the swale ? (This may indicate frequent overtopping of the swale, and as such, blockage of the dam or decreased swale permeability. The dam should be inspected for blockage and the erosion corrected by sodding. There may be a need to provide further erosion control (rip rap, plant stakings) to prevent the re-occurrence of erosion).

5.4.3 Weed Control

Frequency

It is anticipated that weed control may have to be done once every year. Weeds are generally defined as any kind of vegetation which is unwanted in a particular area. In terms of SWMPs, weeds are generally non- native species which tend to decrease the bio-diversity of the planting strategy that was implemented. For example, purple loosestrife would be considered a weed because it is a non-native specie which tends to "out compete" other more sensitive species and create mono-culture planting environments. Dominant species such as purple loosestrife and cattails are generally classified as undesirable species from this perspective.

Methods

Weeding should be done by hand to prevent the destruction of surrounding vegetation. The use of herbicides and insecticides should be prohibited near SWMPs since they create water quality problems. The use of fertilizer should also be limited to minimize the nutrient loadings to the downstream receiving waters.

5.4.4 Plantings

Frequency

Upland and flood fringe plantings are generally stable and should not need much maintenance or re-establishment. Shoreline fringe areas are harder to establish as a result of the frequent wetting and drying associated with this zone. It is estimated that this vegetation may require some re-establishment or enhancement every five years in this zone. Aquatic plantings are the hardest to initially establish. As such, there should be a contingency for the re-establishment of aquatic plantings, and some shoreline fringe plantings, during the first two years of SWMP operation after construction.

Methods

Planting methods can be separated into three main categories based on three major habitat zones provided by the pond treatment system. These are as follows : 1) upland, 2) wet riparian and 3) shallow water.

1) Upland/Flood Fringe

Two types of plantings include ground cover (grasses, herbs) and woody shrubs and trees. Planting should occur in the spring, but after water levels have subsided to a more stable level. Ground cover could be installed either by hydroseeding or using a custom seed mix in a nutrient rich medium impregnated in a geojute biodegradable blanket. Individual shrubs and trees could

be planted manually. In the scenario of the geojute, openings in the material could be made for each planting.

2) Wet Riparian (Shoreline Fringe)

Shoreline fringe plantings should be carried out in mid-May to early June but after the water levels have subsided to a stable level. Some form of protection of the seed mixture (and if a soil nutrient medium is required) should be provided in this dynamic zone of water level fluctuation. The geojute mat suggested in the upland zone is highly recommended in this zone for the establishment of ground cover. Shrubs and trees can be planted through openings created in the mat.

3) Aquatic Fringe/Shallow Water

The establishment of plantings in this zone will require greater materials handling and growth monitoring in the short term and long term. Emergent vegetation is easily planted by hand if the substrate is suitable. Ideally, a firm substrate with at least 10% organics (volume) will allow emergent vegetation to be hand planted. Young shoots, as opposed to rhizomes or corms are preferable for planting as these plants are already growing with an established root structure (for early stability). The plants should be at least 10 cm tall for planting. Planting should occur in late May to early June.

Sprigs or plugs of emergent plant material would be preferable for planting as the root material is already contained in suitable growth medium.

Submerged rooted plants including the pondweeds, should be planted as mature vegetative growth if planted in late spring to early summer. Mature growth will take advantage of warmer water and sunlight penetration. Plantings in early spring or fall should use vegetative propagules such as turions or rhizome plugs which can germinate in spring or overwinter and begin growing in the following growing season.

Water lilies should be planted either directly into the substrate or pre-planted in bio-degradable pots and then installed into the substrate.

Coontail is a floating macrophyte which can be put in the pond at any time in the growing season.

5.4.5 Sediment Removal

Frequency

The frequency of sediment removal depends on many factors :

- upstream land use and level of imperviousness
- active and permanent pool storage (ie. if it is oversized for sediment storage)
- upstream development activities (ie. effectiveness of sediment and erosion control activities)
- SWMP type
- municipal practices (ie. sanding)

These factors were analysed based on the continuous simulation that was performed for end-of-pipe stormwater management facilities. The continuous simulation indicated TSS removal efficiencies for different end-of-pipe stormwater management facilities with varying volumes of storage and different levels of imperviousness. The removal efficiencies were converted into volumes of sediment captured by each type of facility on an annual basis. A set of curves was developed which indicate sediment removal frequency for facility type, storage, and level of upstream imperviousness. These curves are discussed in detail in Section 5.5.

A removal frequency of 10 years is suggested as a minimum design target for end-of-pipe stormwater management facilities.

Methods

The methods for sediment removal depend on the type of SWMP implemented. The following sections describe sediment removal techniques for different types of SWMPs.

Soakaway Pits

Soakaway pits should only be used to infiltrate relatively clean water (rooftops, pervious areas). The potential for clogging these systems is therefore reduced. A plastic mesh or wire mesh filter should be placed in the roof leader system near the ground surface. The roof leader discharge system should have an overflow discharge to the surface. The overflow discharge should be as close to the ground surface as possible to minimize the build-up of head on the soakaway pit. The filter should be located just below the overflow pipe such that overflows will occur if the filter becomes plugged. Frequent overflows during small summer storms will signal that maintenance is required. The filter should be cleaned once a year after the fall period (once the leaves have fallen of the trees).

Grassed Swales

Visual inspection and the aesthetic attributes of swales will indicate the need for maintenance.

Perforated Pipe

Little is known about the maintenance of perforated pipe systems. Most systems which have been implemented have not been maintained. Maintenance of these systems usually refers to the re-construction of these systems. There are several techniques for the maintenance of storm sewers and leachate collection systems that may be applicable to the maintenance of these systems. These will be discussed, but it should be recognized that they are not proven techniques for the maintenance of perforated pipes which convey stormwater, and that monitoring is required to provide a better understanding of the viability of these techniques.

Pre-treatment of the stormwater before it enters the perforated pipe system is fundamental to the longevity of the system. Maintenance activities relating to the pre-treatment system (grassed swales, oversized catch-basins, street sweeping, manhole oil/grit separators), and source control measures (salting and sanding practices) should be implemented to minimize the volume of particulate matter conveyed to the perforated pipe system. Manhole oil/grit separators are beneficial in areas where there are concerns with respect to accidental spills.

In addition, the feasibility of seasonal operation of the system should be investigated since most road systems in Ontario require frequent sanding and salting during the winter. Heavy sand and salt loads will increase the chloride concentration in the surrounding aquifer system and will decrease the longevity of the system by clogging the perforations of the pipe and the void spaces in the surrounding backfill/storage material. The concern with respect to chlorides is prevalent when there is the potential for the exfiltrated water to interact with a deep aquifer system, or if the shallow aquifer system is being used as a supply of drinking water.

Catch-basin cleaning is done via the use of a vacuum truck which extends a hose into the sump of the catch-basin and sucks out the material which has been deposited in the sump. Catch-basin cleaning should be done annually as a general rule, although the municipality should adjust the frequency based on the volume of material removed from the catch-basins during subsequent maintenance operations.

There are three generalized methods for cleaning the perforated pipe system itself :

Flushing — Most municipalities are familiar with sewer flushing. Sewer flushing is generally undertaken to clean out material which has been deposited in the pipe. It is anticipated that clogging will occur at the interface of the perforated pipe and the surrounding backfill/storage if the pipe is not wrapped in filter cloth and at the interface of the pipe and the filter material if the pipe is wrapped in filter cloth. If clogging occurs at the interface of the pipe and the filter material, sewer flushing may not prevent clogging in these systems.

Radial Washing — Radial washing is similar in operation to flushing. The perforated pipe must be connected between manholes and the downstream end plugged or

capped. A water hose is connected to the upstream end of the perforated pipe and water is introduced from the surface into the hose. The perforated pipe is essentially pressurized forcing water out the perforations and hence, cleaning plugged perforations. Radial washing can be performed after flushing if there is considerable sediment deposition in the pipe itself.

Jet Flushing — Jet flushing is frequently used in leachate collection systems for landfills to clean the perforated collection pipes. A pressurized hose is attached to an end nozzle which discharges water in various directions to clean the pipe. The pressure in the pipe on the end nozzle also directs the hose further along the pipe (ie. self directing). There are various nozzle designs available, and one which directs water radially into the perforations would be appropriate for perforated storm sewer applications.

None of these techniques have been extensively tested for stormwater and there will be a need to monitor which method(s), if any, can effectively maintain a perforated pipe system.

Pervious Catch-Basin

Concerns regarding the operation and maintenance of pervious catch-basins are the same as those for perforated pipe systems (see preceding section). The use of pervious catch-basins may be associated with greater concerns, however, since manhole oil/grit separators cannot be implemented to address chemical spills.

Pervious catch-basins will have an over-sized sump which either discharges via a goss trap to an infiltration trench or discharges through a filter bag to the surrounding soils. Both systems require frequent catch-basin cleaning to extend the longevity of the filter bag or infiltration trench. Filter bag systems must be replaced. Few of these systems (filter bags) have been implemented, so the frequency of replacement is uncertain. A general estimate is an annual replacement for the filter bags themselves which can be quite expensive for a large scale development. Given the goss trap entrance to the infiltration trench, maintenance of the trench would not be feasible and these trenches must be reconstructed if they clog.

Infiltration Trench

Infiltration trenches generally consist of trenches filled with gravel, with a sand filter layer underneath the gravel. Water is generally conveyed in to the trench either via underground pipes or overland. The most common form of trenches is the underground variety. Maintenance of these systems generally focuses on ensuring that adequate pre-treatment is provided and that the pre-treatment is operational.

Flushing of pipes in an infiltration trench is generally unfeasible since there are typically several pervious pipes within the trench and cleanout locations would have to be provided at both the inlet and outlet of each pipe length. In addition, flushing may not be effective as discussed previously (see pervious pipe sediment removal). Other than maintaining pre-treatment measures, the only feasible maintenance for infiltration trenches is re-construction.

Infiltration Basin

Infiltration basins are end-of-pipe stormwater management facilities with highly permeable soils. Sedimentation in surface storage facilities such as these will seal the bottom effectively negating the infiltration potential. Maintenance of pre-treatment SWMPs, and the implementation of source controls (salting and sanding practices) are the only methods available to prevent sedimentation from occurring.

Pre-treatment will not be 100% effective in preventing suspended solids from entering an infiltration basin. Tilling the land may be required to maintain the infiltration potential of the soil. In areas where tilling has been tried (Maryland, 1992) little success has been achieved in maintaining the infiltration potential. It is suggested that the planting of deep rooted legumes in an infiltration basin may be beneficial in maintaining the porosity, and hence, infiltration potential in a soil. Consideration must be given, however, to the anticipated growing conditions (frequency of ponding depths) in the basin. Deep basins ($>$ 0.6 m) are discouraged since the weight of water, itself, will tend to compact the soil. Once an infiltration basin has sealed, it is difficult and expensive to remediate, if remediation is possible at all.

Sand Filters

Sand filters can either be surface or subsurface end-of-pipe stormwater management facilities. Surface sand filters may or may not have a grass cover. Sand filters without a grass cover can be raked to prevent clogging and remove trash. Maintenance for grass covered sand filters or subsurface sand filters is similar to infiltration trenches and will focus on ensuring that adequate pre-treatment and source controls are provided.

There are some commercially available sand filters which are primarily targeted for the wastewater and combined sewer overflow (CSO) industry. These may have some application for very small sites and should be investigated for sites where there are land restrictions (ie. these filters can be located in a chamber).

Filter Strips

Maintenance activities for filter strips involve maintaining the vegetated filter, removing sediment from upstream of the level spreader, and ensuring that the level spreader is operating

in accordance with the design. Sediment removal from upstream of the filter strip can be done using a vacuum truck or small grading equipment (Bobcat) if there is considerable sedimentation build-up. Given the small drainage areas serviced by these SWMPs the volume of sediment to maintain/remove will be limited.

Buffer Strips

Buffer strips are generally not engineered and will not provide any location for concentrated collection of sediment. Sediment removal is not proposed for buffer strips since the sedimentation will be dispersed and cleanout would destroy the vegetation and the primary buffering capacity in this area.

Oil/Grit Separators

Manhole oil/grit separators should be cleaned out using a vacuum truck. Some interceptors are designed for CSOs and have a low flow discharge to the sanitary sewer. Although this type of design will facilitate maintenance, it would be undesirable in the case of accidental or deliberate fuel/oil spills since the sewage treatment plant cannot treat large loadings of these pollutants. Therefore it is recommended that any outlets to the sanitary sewer from the oil/grit separator be valved or plugged during everyday operation.

Larger oil/grit separators (tanks) are hard and dangerous to maintain. They usually require manual cleanout using shovels and wheelbarrows. The large oil/grit separators are generally discouraged due to the high capital cost, high maintenance cost, maintenance difficulty, and low removal efficiency of these SWMPs. Three chamber oil/grit separators are supposed to be cleaned out 4 times per year in Maryland. It is suggested that these facilities (3 chamber) be cleaned out twice per year in Ontario and after any known spills have occurred. Manhole separators (dry weather) should be cleaned out annually and after any known spills have occurred (Marshall Macklin Monaghan, 1994).

Wet Ponds, Dry Ponds, and Wetlands

Typical grading/excavation equipment should be used to remove sediment from ponds and wetlands. these generally include Bobcats and backhoes. Dredging is not recommended due to the expensive nature of the undertaking and the potential to destroy features in the facility (ie. vegetation, and bottom grading).

There are several manufacturers who provide remote control robotic vacuums for the maintenance of sewage lagoons. These systems are expensive and infeasible for stormwater ponds unless maintenance is being contracted to a private firm with these resources/expertise.

Frequency of sediment removal for these SWMPs is provided in Section 5.5.

5.4.6 Outlet Valve Adjustment

It is recommended that the extended detention outlets be designed to allow for the adjustment of detention times. This recommendation recognizes that we are still learning about the effects of detention times on water quality enhancement, erosion, and flooding (especially when subwatershed planning has not been undertaken), and that there may need to be operational changes in the field to address site specific, or subwatershed related, concerns on a case by case basis.

Outlet valves can be in the form of sluice gates, globe valves, or gate valves on the end of a reverse sloped pipe. Given the size of most end-of-pipe stormwater management facilities it is likely that sluice gates will be the most economical form of control. Conversely for riser outlets, either a globe valve (instead of an orifice) or alternative orifice plates can be used to manipulate the detention time. The outlet pipe (downstream of the flow control) should always be over-designed to ensure that the orifice/gate/valve acts as the control.

In either type of outlet, the design should incorporate features that would facilitate changing the detention time (ie. access chambers to the outlet, orifice plates which can be slid in and out of a riser, etc.).

5.4.7 Trash Removal

Trash removal is an integral part of SWMP maintenance. Generally there will be a need to undertake a "spring cleanup" with respect to trash removal for all surface SWMPs. Trash removal is then performed as required based on the observations of trash build-up during the regular inspections.

5.5 Pond and Wetland Sediment Removal

To ensure long term effectiveness, the sediment that accumulates in wet ponds, wetlands and dry ponds should be periodically removed. The required frequency of sediment removal is dependent on the type of SWMP, the design storage volume and the characteristics of the upstream catchment.

The accumulation of sediments results in a reduction of the storage volume of the SWMP. This reduction of storage volume will reduce the long term suspended solids removal efficiency of the SWMP. The required maintenance frequency for sediment removal can be determined based on the rate of performance reduction with loss in storage volume. This methodology is subjective since an allowable loss in performance must be accepted, after which the facility must

be maintained. In addition, the performance-storage relationship is based on theoretical calculations with many assumptions. These calculations do not account for conditions such as upstream development and poor sediment and erosion controls. As a result, the maintenance frequencies derived in this analysis provide a starting point which should be refined based on operational and maintenance experience in the field.

The performance-storage curves become asymptotic quickly (ie. once the curve starts to become asymptotic, a large increase in storage is required to improve the removal performance). Accordingly, under these conditions there must be a considerable loss in storage to reduce the effectiveness of the facility by 10% for the typical volumes of storage that will be implemented. Therefore, it was assumed that an acceptable reduction of TSS removal efficiency, due to gradual sediment accumulation, would be 5%.

The average annual TSS removal efficiency of a SWMP with a certain volume of storage was determined using continuous simulation and a sedimentation model (ie. the results to determine the performance curves in Chapter 4).

The required maintenance frequency for this SWMP was then determined based on the annual sediment accumulation and resulting annual loss in storage. The timeframe to reduce the storage to the point that the annual removal efficiency was 5% less than the original efficiency indicates the maintenance frequency for that SWMP with that particular storage.

If excess storage is provided to lengthen the intervals between maintenance the timeframe to reduce the efficiency by 5% below the <u>original</u> efficiency should be calculated. For example, if 80% removal is required, but excess storage is provided resulting in a initial efficiency of 85%, then maintenance would be required when the performance efficiency was reduced by 10% (ie. 5% below the original target efficiency).

Curves of maintenance frequency by SWMP type, storage, and different levels of upstream imperviousness were calculated based on the continuous simulation results and the requirement for maintenance with a 5% loss in TSS removal performance. Plots of these curves indicated that there is a linear relationship between maintenance frequency and SWMP storage. This linear relationship is shown in Figures 5.1 through 5.4. The straight lines in these figures are best-fit lines based on linear regression. Figures 5.1 through 5.4 only display maintenance frequencies up to a maximum of 50 years. It is anticipated that any comparison between SWMP options would not be based on a timeframe longer than 50 years recognizing the rapid evolvement of this discipline.

Figures 5.1 through 5.4 can be used to determine the required sediment removal frequency given the SWMP type and storage volume.

The figures also indicate the longer maintenance interval given additional storage. This is somewhat deceiving, however, since these curves represent the maintenance frequency for a 5% reduction in performance. If additional storage is provided, the maintenance frequency would

be based on a larger reduction in performance. In order to allow users to calculate the required maintenance frequency for oversizing a SWMP, annual suspended solids loadings in runoff from catchments with different levels of imperviousness, and estimated sediment density, are provided in Table 5.3. The values of suspended solids loadings in Table 5.3 were derived from SWMM simulation results and are intended to be used as estimates for planning purposes only. The density of suspended solids was based on a review of the literature of stormwater sediment characteristics and recent pond sediment removal data.

The following methodology should be used to calculate the maintenance frequency if the SWMP storage is oversized :

1. Determine the appropriate TSS removal efficiency based on Habitat Type
2. Subtract 5% to obtain the target maintenance removal efficiency
3. Determine the projected TSS removal efficiency based on the storage provided
4. Calculate the loss in removal performance and loss in storage for each year based on the removal performance at the start of the year, the suspended solids loading rate, and the sediment density. The removal efficiency at the start of the next year will be based on the resulting available storage volume at the end of the year. These calculations are continued until the removal efficiency of the facility at the start of the year is equal to the target maintenance removal efficiency. This calculation can be easily automated in a spreadsheet format. (Given the large maintenance intervals a conservative estimate would be to assume a constant removal rate each year equal to the initial removal rate such that only one calculation has to be performed. Following this method, a linear calculation is made to determine how long it takes to accumulate the difference in storage volumes between the initial storage and the target maintenance storage volume)

Table 5.3 Annual Sediment Loadings

Catchment Imperviousness	Annual Loading (kg/ha)	Wet Density (kg/m^3)	Annual Loading (m^3/ha)
35%	770	1230	0.6
55%	2300	1230	1.9
70%	3495	1230	2.8
85%	4680	1230	3.8

Fig. 5.1 Storage Volume vs. Removal Interval
(for 35% Impervious Catchments)

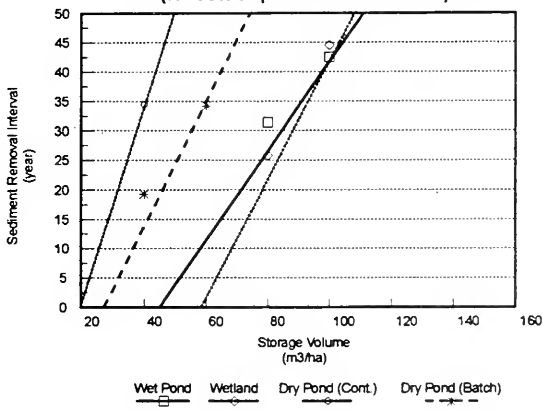


Fig. 5.2 Storage Volume vs. Removal Interval
(for 55% Impervious Catchments)

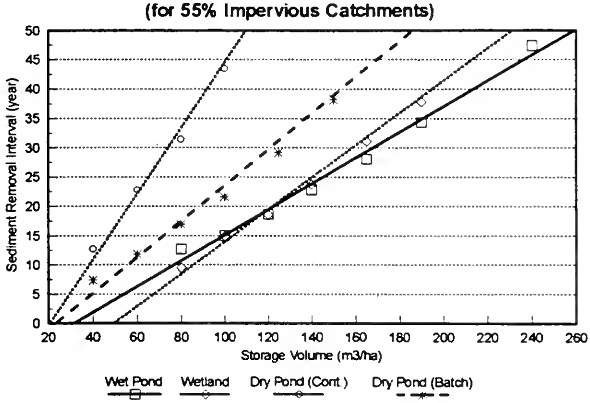


Fig. 5.3 Storage Volume vs. Removal Interval
(for 70% Impervious Catchments)

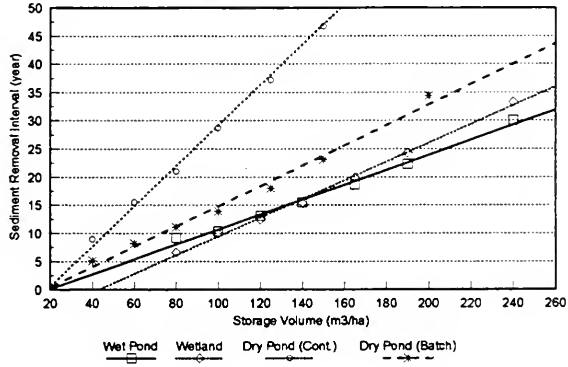


Fig. 5.4 Storage Volume vs. Removal Interval
(for 85% Impervious Catchments)

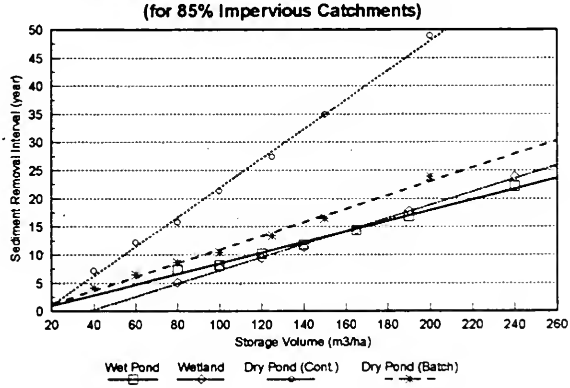


Table 5.3 provides allows a conservative estimate of annual sediment accumulation in a SWMP by multiplying the annual loading of suspended solids (m^3/yr) by the long term removal efficiency for the particular SWMP, level of imperviousness, and storage provided.

5.5.1 Sediment Disposal

Generally the sediment which is removed from SWMPs will not be contaminated to the point that would be classified as hazardous waste. All sediment which is removed from SWMPs should be tested, however, to determine the disposal options. The MOEE produces sediment disposal guidelines which should be consulted for information pertaining to the exact parameters and acceptable levels for different disposal options. Most private laboratories are familiar with the disposal guidelines and can test sediment samples with these in mind.

There are three generalized disposal options :

On-site Disposal

On-site disposal allows the sediment to be disposed of on any land area that is not regulated (ie. floodplain, lakes, etc.). In the planning stage of land requirements for subdivision/site plan stormwater management requirements, land can be set aside for on-site disposal of sediments which are removed from the various SWMPs that are implemented. The areas that are used for sediment disposal should be landscaped to provide a natural appearance after each sediment removal operation.

Off-Site Disposal

It is anticipated that off-site disposal may be preferred by most developers and municipalities since off-site disposal does not reduce the developable area, landscaping/grading does not have to be performed, and there are no perceived liability/health concerns with respect to the surrounding landowners. Off-site disposal can mean disposal at a sanitary landfill or disposal at another area undergoing filling. The decision of where the material is deposited depends on the quality of the sediments and the availability and distance of the opportunity fill area.

Temporary disposal areas are recommended for surface end-of-pipe stormwater management facilities that do not have a maintenance by-pass since it provides a location for the sediment to dry before transporting it off-site.

Hazardous Waste

Although the sediment is expected to be contaminated (high metals, bacteria, nutrients) there has not been any reported cases where the material could be classified as hazardous waste. Hazardous waste must be deposited at a hazardous waste facility (there is one operated by Tricil

in Sarnia). The transportation costs and disposal fees are expensive for hazardous waste since licensed haulers must be used to transport the material and there are relatively few facilities in the province. It is unlikely that any removed sediments from stormwater will be classified as hazardous waste.

5.6 Winter Operation

The climate in Ontario restricts the performance of SWMPs during the winter and spring periods. Frozen or saturated ground conditions increase the rate of surface runoff during the spring freshet. Snowmelt combined with rainfall creates large flows (especially in rural areas) which cannot be accommodated by SWMP systems. Additional pollutant loadings (salting and sanding for public safety on the road network) can also be expected during this period as a direct result of the climatic conditions.

The largest impacts of winter operation are perceived to occur for stormwater measures which promote infiltration. Winter operation of infiltration SWMPs that accept road runoff may have adverse impacts (increased chloride loadings to interflow or groundwater) and may decrease the longevity of these SWMPs (from high sediment loadings).

It is therefore suggested that any infiltration SWMP which accepts road runoff from areas subject to frequent sanding and salting (high traffic areas) be designed and constructed with a by-pass which could be operated during the winter and spring season to prevent the clogging of the facility and to prevent the transmission of chlorides to the groundwater system. The by-pass could be initiated several weeks before the average annual date of the first frost and stopped after the spring freshet (ie. the snow has melted). In areas with curb and gutter servicing, the road system should be swept before the infiltration system is re-established. In areas with ditch and culvert, the ditches should be inspected and cleaned as required before the infiltration system is re-established.

Although suggested, the use and operation of a winter by-pass is left to the discretion of the municipality. The winter operation of road runoff infiltration measures should be noted on the design submission for a Certificate of Approval.

Infiltration systems which do not accept road runoff, or accept road runoff from local streets that are not subject to sanding and salting, would not be subject to these complications. Designers of SWMP systems, however, should be wary of frost heave and its affect on underground infiltration systems (trenches, soakaway pits). Guidance is provided in Chapter 3 to identify sites with soils that might be susceptible to frost heave, and to indicate the depth of soil cover required based on the infiltration storage volume and depth. Generally soils with a silty composition will be subject to frost heave.

5.7 Maintenance Enhancements

It is important for SWMP planners and designers to ensure that any designs facilitate maintenance activities. Many of these enhancements are described in Chapter 3 in detail. Key aspects of a SWMP design which should be reviewed with respect to maintenance include :

- access

  A maintenance route should be established to allow vehicles to gain access to the SWMP. This applies to all measures except stormwater lot level controls. It must be expected that access to stormwater lot level controls may not be possible given the tendency for homeowner's to construct fences, gardens, landscaping, etc. If stormwater lot level or conveyance controls (ie. enhanced swales or trenches) are proposed along the rear lot lines, municipalities can obtain an easement for maintenance. The logistics of maintaining access to the easement will require considerable diligence/effort on the part of the municipality and may not be feasible.

  The slope of the access route should be reviewed to ensure that it is conducive for maintenance vehicles (ie. 4:1 or flatter).

  Access to inlet and outlet structures, flow splitters, and by-pass manholes/chambers is also important. Access to an outlet structure for a pond or wetland can be provided by placing the outlet in a chamber in the embankment. Locating the outlet in a chamber enhances the aesthetics of the SWMP and reduces the potential for vandalism.

- forebays

  Forebays are applicable for most end-of-pipe stormwater management facilities (wetlands, wet ponds, dry ponds, infiltration basins). Forebays allow the area of maintenance to be concentrated to one location thereby facilitating maintenance operations. Forebays have a permanent pool which should be drawn down for maintenance. Given that the bottom of a forebay will still be saturated after drawdown, it should be lined with open block/stone below the permanent pool to ensure that vehicles do not get stuck in soft material.

  Consideration must be given to the berm between the forebay and the rest of the facility. In cases where the forebay releases to a dry pond or infiltration basin, a gravity drainable pipe (if physically feasible) can be installed in the berm to draw down the forebay. In cases where the forebay releases to a wet pond or wetland, there are two options. The water level in the downstream portion of the facility can be lowered until the berm is emergent. Water can then be pumped from the forebay to the downstream portion of the facility until the forebay is

dry. In this scenario the berm must be designed as a small dam if the water in the downstream portion of the facility is not completely drained. Maintaining water in the downstream portion of the facility has the benefit of reducing the impacts to the aquatic and shoreline fringe vegetation. The second option would be to drain both facilities. This could be accomplished by either valved gravity draining maintenance pipes (if feasible) in both the forebay and the downstream portion of the facility, or by pumping if the facilities cannot be gravity drained.

■ maintenance/drawdown pipe

In surface storage facilities (wet pond, wetland, dry pond, infiltration basin), a maintenance pipe should be provided to draw down the permanent pool for maintenance. This maintenance pipe should be set near or at the bottom of the facility. If a gravity drainable pipe is not feasible the facility will have to be pumped when maintenance is required. If possible, the pond should be drawn down early in the morning or overnight to reduce downstream thermal impacts. A geotextile filter bag should be attached to the end of the maintenance pipe to prevent the discharge of contaminants from the facility into the receiving waters.

■ pre-treatment

Adequate pre-treatment (oil/grit separators, roof leader filter traps, grassed swales) should be provided for infiltration SWMPs. These types of measures are described in Chapter 3.

■ maintenance by-pass

Maintenance may take several days to a week to perform. Storms during this time should be routed around the SWMP. The by-pass should be located either at the inlet to the SWMP or slightly upstream of the SWMP. In piped systems this is accommodated by fitting sluice gates to the by-pass pipe and SWMP inlet pipe in an upstream manhole. For maintenance operations, the gate to the SWMP can be closed and the gate to the by-pass pipe opened.

This type of system can also be used for the seasonal operation of infiltration systems that accept road runoff.

■ over-sizing SWMP storage

Over-sizing the storage provided in a SWMP compared to what is required to achieve performance targets will decrease the maintenance frequency in a SWMP. It is left to the discretion of municipalities to set requirements for maintenance frequency, if desired, in addition to the provincial/municipal water management requirements.

- sediment disposal areas

  Sediment removal operations and costs can be reduced if areas are set aside for sediment disposal. These areas can be used for either permanent sediment disposal or temporary disposal (to allow the sediment to dry before transporting off-site for permanent disposal). Areas for temporary disposal are recommended for surface end-of-pipe stormwater management facilities that do not have a maintenance by-pass.

6.0 CAPITAL AND OPERATIONAL COSTS

6.1 Application of Costing Information

This chapter provides information with respect to capital and operations and maintenance costs for stormwater Best Management Practices. The cost of urban stormwater solutions is important to the overall economic health of development and should not be overlooked when assessing alternative stormwater management measures. Maintenance and operation costs are, arguably, the most important since these facilities are to be maintained and operated in perpetuity.

The information in this chapter is to be used for planning purposes only. **Site specific costing should be determined in all cases** where urban stormwater management practices are contemplated since there are many site specific conditions that will affect the real cost of implementation.

The planning of SWMPs involves investigating different SWMP alternatives at a site, subdivision, or subwatershed level. The costs in this chapter can be used to assess the economical impacts of various SWMP solutions and combinations of different SWMP solutions. It is anticipated that the information in this chapter will be most useful at a plan of subdivision level and subwatershed level, since site specific information will probably be available at the site plan level to provide more accurate costing estimates.

The total cost of a SWMP includes capital costs, the present value of operation and maintenance costs, engineering costs, and contingency costs. Each of these costs is discussed in subsequent sections of this chapter.

6.2 SWMP Capital Costs

The capital cost represents the estimated construction cost of the stormwater facility(s). This will include costs for excavation, cutting and filling, grading, structures, fittings, environmental site controls (sediment and erosion controls), and material as required.

The required types of construction and materials for various end-of-pipe stormwater management facilities are indicated in Table 6.1. Table 6.1 provides a shopping list of items to prepare an estimate of the capital cost of a SWMP(s). Several variations of items (reverse sloped outlet pipe, riser outlet pipe) have been included to provide a better estimate of the cost for different SWMP configurations. Accordingly, duplicate items should not be costed in Table 6.1 (ie. cost for a reverse sloped outlet pipe or a perforated riser outlet pipe).

Table 6.1 Capital Cost Items for End-of-Pipe Stormwater Management Facilities

Type of Construction or Material	wet pond	wetland	dry pond	infiltration basin	infiltration trench	soak away pit	filter strip	sand filter	oil/grit* separator	grass swales	flow splitter
Excavation (off-site disposal)	X	X	X	X	X	X	X	X		X	X
Earthwork (cut and fill on-site)	X	X	X	X	X	X	X	X		X	X
Erosion block/stone	X	X	X								
Concrete Outlet Structure	X	X	X								
Concrete Outlet Pipe	X	X	X								
Perforated Riser Outlet	X	X	X								
Perforated Riser Outlet Trash Rack	X	X	X								
Infiltration Observation Well					X						
Rip Rap	X	X	X							X	
Perforated Pipe				X	X	X	X	X			
Seed and Topsoil	X	X			X						
Clear Stone (gravel)					X	X		X			
Filter Cloth					X	X		X			
Filter Material (sand)					X			X			
Submergent and Emergent Vegetation	X	X									
Shoreline Fringe and Flood Fringe Vegetation	X	X	X								

Table 6.1 Capital Cost Items for End-of-Pipe Stormwater Management Facilities (continued)

Type of Construction or Material	wet pond	wetland	dry pond	infiltration basin	infiltration trench	soak away pit	filter strip	sand filter	oil/grit* separator	grass swales	flow splitter
Upland Vegetation	X	X	X	X			X				
Temporary Fencing (Post and Wire)	X	X									
Grass Sod and Topsoil			X	X			X	X		X	
Concrete (Poured in place)									X		X
Trash Rack (metal)									X		
Inverted Elbow Pipe									X		
Outlet Valve/Gate Controls	X	X	X								X

X usually required
* 3 chamber oil/grit separator

6.2.1 Pre-Treatment SWMPs

In addition to the items listed in Table 6.1, some of the SWMPs listed in the table require pre-treatment to ensure proper operation and longevity. Pre-treatment is required for infiltration SWMPs which accept road runoff (ie. perforated storm sewers, end-of-pipe infiltration trenches, end-of-pipe infiltration basins) since road runoff contains a high suspended solids load that can quickly clog these types of systems.

In order to provide pre-treatment, storage (wet pond, wetland, dry pond) and/or vegetative SWMPs are generally implemented upstream of the SWMP that requires protection. Although the size of the pre-treatment SWMPs will depend, to a certain extent, on the land available, the size can be estimated using the sizing rules provided in Chapter 3. The size of the storage SWMPs should be based on the forebay sizing rules, and not on providing full water quality treatment. Table 6.2 provides a list of SWMPs that require pre-treatment, and possible pre-treatment SWMPs. In the case of surface end-of-pipe stormwater management facilities, pre-treatment SWMPs represent forebays.

Table 6.2 Pre-Treatment SWMPs

SWMP	Need for Pre-treatment	Pre-treatment SWMPs				
		Grassed Swales	Filter Screen	Filter Strip	Oil/Grit Separator	Wet Pond
Wet Pond	⊠	X				X
Wetland	⊠	X				X
Dry Pond	⊠	X				X
Infiltration Basin	■	X		X	X	X
Infiltration Trench	■	X		X	X	X
Pervious Pipe	■	X			X	
Pervious Catch-Basin	■	X				
Soakaway Pit	■		X			
Filter Strip	⊠	X				X
Sand Filter	⊠	X		X	X	X
Oil/grit Separator	□					
Grassed Swales	□					

■ Pre-treatment required
⊠ Pre-treatment enhances performance
□ Pre-treatment not required

Table 6.3 Unit Costs for Capital Construction		
Type of Construction or Material	Unit	Price
Land Alterations		
Excavation (off-site disposal)	m^3	$ 10
Earthwork (cut and fill on-site)	m^3	$ 3
Construction Materials		
Erosion block/stone	m^2	$ 50
Concrete Outlet Structure	each	$ 5500
Concrete Outlet Pipe (300 mm / 600 mm / 900 mm)	m	$ 70 / $ 170 / $ 300
Observation Well (100 mm PVC)	each	$ 15
Rip Rap (450 mm)	m^2	$ 50
Perforated Pipe (100 mm, plastic)	m	$ 10
Perforated Riser Outlet Pipe (300 mm, plastic)	m	$ 90
Perforated Riser Outlet Trash Rack (400 mm CMP)	m	$ 100
Temporary Fencing (Post and Wire)	m	$ 15
Concrete (poured in place)	m^3	$ 600
Trash Rack (metal)	m^2	$ 100
Inverted Elbow Pipe	each	$ 300
Outlet Gate Valves (300 mm/600 mm)	each	$ 1200 / $ 4800
Outlet Sluice Gates (300 mm / 600 mm / 900 mm)	each	$ 5500 / $ 8000 / $11500
Clear Stone (gravel, 25 mm ~ 50 mm)	m^3	$ 35
Filter Cloth	m^2	$ 1
Filter Material (sand)	m^3	$ 15
Vegetative Plantings		
Seed and Topsoil	m^2	$ 2.5
Grass Sod and Topsoil	m^2	$ 4.5
Emergent and Submergent Vegetation	m^2	$ 12
Shoreline Fringe and Flood Fringe Vegetation	m^2	$ 12
Upland Vegetation	m^2	$ 5
Trees (Wooded Filter Strips)	m^2	$ 25

Unit prices for different types of construction activities and material that are required to implement various SWMPs are listed in Table 6.3. The unit prices in Table 6.3 are estimates under normal construction circumstances only, and include labour. Local or site and project specific estimates should be made whenever possible.

6.3 SWMP Operations and Maintenance Costs

The operation and maintenance costs represents the costs to ensure the proper operation, longevity, and aesthetic functioning of the stormwater control measures. The necessary tasks to achieve these objectives include sediment removal, trash removal, maintenance of the vegetation, and inspections of the inlet(s) and outlet(s).

The costs associated with urban SWMP operation and maintenance vary with SWMP type and size. Different types of SWMPs require different types of maintenance activities. Table 5.1, in Chapter 5, lists the commonly required maintenance activities for the urban SWMPs discussed in this manual.

As mentioned in Chapter 3, infiltration SWMPs have the shortest recorded longevity of any SWMPs that have been implemented based on the monitoring that has been conducted to-date. Sediment removal for these SWMPs represents complete re-construction. No pre-treatment measure can remove 100% of the suspended solids from the influent discharge to an infiltration SWMP. Therefore, it is not a matter of if they will clog, but when they will clog. Accordingly, in the determination of maintenance costs for comparison purposes, entire SWMP re-construction should be considered for infiltration type SWMPs based on an estimated longevity.

Estimations of the longevity of these SWMPs are based on professional opinion. Guidance is provided in Equation 6.1 and Table 6.4 for estimating the longevity based on literature monitoring results and the native soil permeability. Recognizing the subjectiveness of equation 6.1, there will be flexibility in assessing the longevity of these SWMPs based on site specific information.

The estimation of infiltration longevity assumes that the water table and bedrock conditions at the site are suitable for infiltration. These conditions must be confirmed since they will also have considerable impact on the longevity/operation of infiltration SWMPs.

$$L = (P \times T)^{0.4}$$ Equation 6.1 Longevity of Infiltration SWMPs

where L = longevity (years)

P = permeability (mm/h)
T = longevity factor from Table 6.5 (years)

Table 6.4 Estimated Infiltration SWMP Longevity	
Infiltration SWMP	Longevity Factor
Soakaway Pit	60
Infiltration Basin	15
Infiltration Trench	25

The values in Table 6.4 assume that there is adequate pre-treatment upstream of the infiltration SWMPs. Without adequate pre-treatment the expected useful life of an infiltration SWMP is considerably shorter than that given in Table 6.4 (approximately 5 years).

Table 6.5 provides a list of unit prices for the operations and maintenance activities listed in Table 5.1. The unit prices do not include transportation costs or the costs for equipment to perform the maintenance (ie. bobcats or front end loaders for sediment removal). It was assumed that the owner of the stormwater management works would be the local municipality, and that they would have the required equipment as part of their works department. The unit prices in Table 6.5 include labour and represent typical maintenance conditions (ie. draw down wet SWMPs to maintain in the dry). Other maintenance activities such as dredging should be costed on a site specific basis.

Table 6.5 was prepared to provide planning level estimates of long term SWMP costs. Site specific maintenance and operations costs should be calculated wherever possible.

Table 6.5 indicates that the frequency of sediment removal depends on the SWMP type and design storage volume. The frequency of sediment removal can be determined using Figures 5.1 to Figure 5.4 which are presented in Chapter 5.

Table 6.5 Unit Costs for Operations and Maintenance			
Type of Maintenance	Maintenance Interval (yrs)	Unit	Price
Litter Removal	1	ha	$ 2000
Grass Cutting	***	ha	$ 250
Weed Control	1	ha	$ 2500
Vegetation Maintenance (Aquatic/ Shoreline Fringe)	5	ha	$ 3500
Vegetation Maintenance (Upland/ Flood Fringe)	5	ha	$ 1000
Sediment Removal (front end loader)	*	m^3	$ 15
Sediment Removal (vacuum truck - catch-basin, filter strip, grassed swale)	*	m^3	$ 120
Sediment Removal (manual - oil/grit separator, sand filter)	*	m^3	$ 120
Sediment Testing (lab tests on quality)	*	each	$ 365
Sediment Disposal (off-site landfill)	*	m^3	$ 300
Sediment Disposal and Landscaping (on-site)	*	m^3	$ 5
Inspection (Inlet/Outlet, etc.)	1		$ 100
Pervious Pipe cleanout (flushing)	5	m	$ 1
Pervious Pipe cleanout (Radial Washing)	5	m	$ 2
Seasonal Operation of Infiltration By-pass	0.5	**	$ 100
Infiltration Basin Floor Tilling & Re-vegetation	2	ha	$ 2800

* frequency of sediment removal depends on SWMP type and volume
** dependent of infiltration facility (based on centralized facility) Seasonal operation of a system with many inlets (ie. pervious pipe system) would be more expensive.
*** no grass cutting or minimal frequency of grass cutting (once or twice per year)

6.4 Engineering and Contingency Costs

Both the engineering costs and contingency costs will vary significantly from site to site and project to project. Planning level costs can be assigned to each of these components based on experience in the consulting practice.

6.4.1 Engineering Costs

The engineering costs include the planning, design and construction of all stormwater management works/measures. For planning purposes, the engineering cost is based on the total capital cost of the stormwater works to be implemented. As a simple rule of thumb, the engineering cost of a SWMP can be estimated as 10% of the total construction cost for that SWMP.

6.4.2 Contingency Costs

Contingency costs represent the unforeseen costs that may occur during the construction of a SWMP. These may include additional construction costs (ie. bedrock excavation, dewatering, etc.), additional material costs, and design alterations. As such, actual contingency cost will vary significantly from project to project. Generally the contingency cost is estimated as 15% of the total construction cost for a given project.

In the case of stormwater management works that require maintenance (ie. facilities that are designed for either water quality enhancement or erosion control using extended detention with a drawdown time ≥ 12 hours), however, the contingency cost should be estimated as 15% of the total of the construction cost and the present value of operations and maintenance costs.

6.5 SWMP Overall Cost Calculation

The total capital cost and total maintenance cost of a SWMP can be calculated using Tables 6.1 through 6.5. The following procedure provides a stepwise methodology to calculate the present value of the capital and maintenance cost of a SWMP solution for comparison with other SWMP solutions.

Step 1 Review Table 6.1 to ensure that all capital cost items for a selected SWMP type have been identified.

Step 2 Review Table 6.2 to identify the need for pre-treatment SWMPs based on the type of SWMP selected for implementation. Review Table 6.1 to identify the capital cost items required for the pre-treatment SWMP.

Step 3 According to the preliminary design of the selected SWMP type, and pre-treatment SWMP requirements, estimate the required quantities of each of the capital cost items (including any site specific requirements not identified in Table 6.1) identified in Steps 1 and 2.

Step 4 Use Table 6.3 (unit prices) to calculate the capital cost of each item identified in Steps 1 and 2 (capital cost is the product of the unit price and the required quantity), and then calculate the total capital cost as the sum of the costs of all the required capital cost items.

Step 5 Review Table 5.1 to identify the operations and maintenance activities that are required for the selected SWMP.

Step 6 Use Table 6.5 to identify the required maintenance interval and the unit price for each of the required operation/maintenance activities. In the case of sediment removal, refer to Chapter 5 to determine the required sediment removal interval.

Step 7 Estimate quantities for the required operation/maintenance activities according to the preliminary design of the selected SWMP; and then calculate the maintenance cost for each activity. The quantity of accumulated sediments, and sediment removal frequency can be estimated using Figures 5.1 to 5.4 in Chapter 5.

Step 8 Group the operations and maintenance costs for activities that are performed numerous times per year into an annual maintenance cost. Sum up other operations/maintenance costs that have the same frequency of occurrence (ie. 2 year, 5 year, 10 year, etc.). The summations should include re-construction activities for infiltration SWMPs where necessary based on Table 6.4 and Equation 6.1.

Step 9 Calculate the present value of operations/maintenance activities for similar frequency occurrences. Equation 6.2 can be used to calculate the present value of these activities based on an annual interest rate and service life of the SWMP.

$$PV = \sum_{t}^{T} [\, OM \times (1+r)^{-t} \,] \qquad \text{Equation 6.2 Present Value}$$

where :

OM	=	Sum of operations/maintenance costs that are required to be performed every t years
t	=	the interval between maintenance activities in years (interval between maintenance is equal to the step of the summation; ie. if the interval is 3 years the summation proceeds with t= 3, then t = 6, then t = 9...etc. until t = T)
r	=	annual interest rate.
T	=	the service life of the selected SWMP

Equation 6.2 is best utilized in a simple spreadsheet format. The service life and interest rate are user defined. Typical values would be a 50 year service life and 3% interest rate (interest rate should be discounted to account for inflation, ie. 8% interest rate - 5% inflation rate = 3% interest rate).

Step 10 Add the total capital cost (plus engineering cost) to the sum of all the present values for operations/maintenance activities. The contingency cost should then be added to the resultant number to obtain the total present value of the cost of implementing the SWMP.

The value determined in step 10 should be used for comparison purposes with other SWMP solutions to indicate the difference in economic impacts of choosing one SWMP solution over another.

6.6 Land Requirements of End-of-Pipe Stormwater Management Facilities

End-of-pipe stormwater management facilities (wet pond, dry pond, wetland and infiltration basin) require the use of land which might otherwise be available for development (since SWMPs should be located on the tableland and not in the floodplain). It is important to recognize this when estimating the area of developable land and the area required for the implementation of these types of SWMPs. The land costs for different SWMP solutions can be added to the value determined in step 10 of the overall calculation of costs (Section 6.5) for comparison purposes if the cost of the land is known. Land costs are extremely variable, and as such, the cost of land is subject to dispute. Site specific data must be used in the analysis for land costs to have any real meaning.

The cost of land depends on the location and size. The area of land required by an end-of-pipe stormwater management facility depends on the design storage volume, the side slopes and its shape. The following sections provide methods to estimate the area of land required by a SWMP. It is stressed that the following equations were derived using specific assumptions concerning the side slopes and shape of the various SWMPs in order to simplify the analysis. Derivations for the equations that are provided in the following sections are given in Appendix I. The results from these calculations will only result in planning level information which can be used to compare SWMP concepts. The actual size of the SWMP will depend on the existing topography, servicing options, and surrounding natural features.

6.6.1 Wet Ponds and Wetlands

The following configuration was assumed to be indicative of typical design parameters for wet ponds and wetlands :

- bottom of the wet pond/wetland was assumed to be rectangular in shape

- length to width ratio of 3:1
- side slopes of 4:1 within the permanent pool
- side slopes of 5:1 in the extended detention portion of the pond/wetland

Based on these assumptions the area of land required by a wet pond or wetland can be estimated as follows :

Step 1 Determine the width of the bottom of the pond or wetland (X in metres)

$$X = \frac{\sqrt{256h_p^4 - 12h_p(\frac{64}{3}h_p^3 - PV)} - 16h_p^2}{6h_p}$$

Equation 6.3 Wet Facility Bottom Width

where : hp - the average depth of the permanent pool (m)
PV - the permanent pool volume (m^3)

Step 2 Determine the depth of the extended detention in the wet pond/wetland (h_e in metres)

$$h_e = \frac{\sqrt{(X+8h_p)^2(3X+8h_p)^2 + 20(3X+8h_p)EV}}{10(3X+8h_p)} - \frac{X+8h_p}{10}$$

Equation 6.4 Active Storage Depth

where : EV - extended detention volume (m^3)

Step 3 Determine the area of land required for the wet pond or wetland (LA in m^2)

$$LA = (X+8h_p+10h_e)(3X+8h_p+10h_e)$$

Equation 6.5 Wet Facility Area Requirement

6.6.2 Dry ponds and Infiltration Basins

The same calculation method can be used to estimate the land area required by dry ponds and infiltration basins. The following configuration was assumed to be indicative of typical design parameters for dry ponds and infiltration basins :

- bottom of the pond/basin was assumed to be rectangular in shape
- length to width ratio of 3:1
- side slopes of 5:1 in the extended detention portion of the pond/wetland

Step 1 Determine the width of the bottom of the pond/basin (X in metres)

$$X = \frac{\sqrt{400h^4 - 12h\left(\frac{100}{3}h^3 - EV\right)} - 20h^2}{6h}$$

Equation 6.6 Dry Facility Bottom Width

where : EV - the design extended detention volume (m^3)
h - the average depth of the extended detention storage (m)

Step 2 Determine the area of land required for the infiltration basin or dry pond (LA in m^2)

$$LA = (X+10h)(3X+10h)$$

Equation 6.7 Dry Facility Land Requirements

It should be noted that the area of land required is not linearly related to the design storage volume of the SWMP. Accordingly, extrapolation should not be used to calculate the land area in cases where different design storages are considered.

6.6.3 Acceptable Ranges of Design Parameters

The calculation of land area in Section 6.6.2 requires the average permanent pool depth (wet facilities) and active storage depth (dry facilities) as inputs. Acceptable ranges of these design parameters are summarized in Table 6.6. Detailed discussions on the requirements of the design parameters of various end-of-pipe stormwater management facilities are provided in Chapter 3.

Table 6.6 Acceptable Ranges of Design Parameters

	Wet Pond	Dry Pond	Wetland	Infiltration Basin
Permanent Pool Depth	1 to 3 m		0.15 to 0.30 m	
Extended Detention Storage Depth	1 to 1.5 m	1 to 3 m	≤ 1 m	≤ 0.6 m

7.0 REVIEWER'S CHECKLIST

This checklist is a summary of key design parameters/considerations that has been provided as guidance to both reviewers as well as designers in previous chapters of the manual. It is not an exhaustive checklist, nor is it intended as a mandatory list of requirements. A list of requirements for a stormwater management report can be found in a recent document prepared for the Provincial Facilitator's Office entitled "A Guide to the Current Stormwater Management Plan Review Process" (Office of the Provincial Facilitator, 1994).

7.1 Stormwater Lot Level Controls

Reduced Lot Grading

(1) The site should be naturally flat. (ie. excessive grading is not required to implement reduced lot grading).
(2) There should be an apron of 2 to 4 metres around the building at a higher grade (ie $\geq$ 2%) to prevent foundation drainage problems.
(3) The major system flow path should be well defined.
(4) Roof leaders discharging to the surface should extend 2 m away from the building.
(5) The pervious area depression storage can be increased by 0.5 mm for every 0.5% decrease in lot grading from 2 %.
(6) Tilling of the first 300 mm of soil can be used to increase permeability and reduce compaction.

Roof Leader to Ponding Area

(1) The maximum ponding depth should be $\leq$ 100 mm.
(2) The major system flow path for flows in excess of the ponding depth should be well defined.
(3) The ponding area should be $\geq$ 4 m away from any building foundation to prevent foundation drainage problems.
(4) The ponding area should accommodate at least the first 5 mm over the roof top area.
(5) The ponding area should not accommodate > 20 mm over the roof top area.
(6) The soils should be suitable for infiltration (permeability $\geq$ 15 mm/h)
(7) Tilling of the first 300 mm of soil can be used to increase permeability and reduce compaction.
(8) The maximum ponding depth on commercial and industrial roofs with no outlet would be 10 mm. The maximum ponding depth on commercial and industrial roofs during the 100 year storm should be $\leq$ 65 mm.

Roof Leader Discharge to Soakaway Pits

(1) The soils should be suitable for active infiltration (≥ 15 mm/h percolation rate).
(2) The soakaway pits should be located ≥ 4 m away from any building foundation to prevent foundation drainage problems.
(3) The storage volume should accommodate at least the first 5 mm over the roof top area.
(4) The storage volume should not accommodate > 20 mm over the roof top area.
(5) The depth of the soakaway pits should be based on the soil stratification. If there is an underlying sand layer or the soils are greatly horizontally stratified, deep pits may be acceptable, otherwise shallow pits should be contemplated (≤ 1.5 m deep).
(6) The depth of cover over the soakaway pit should account for any potential frost heave. Poorly graded native soils with a high silt content are susceptible to frost heave.
(7) An overflow pipe should be installed from the roof leader to discharge to a splash pad. The overflow height should be located as close to the top of the soakaway pit as possible to minimize the build-up of head on the pit before overflow occurs.
(8) A removable filter should be incorporated into the roof leader below the overflow pipe.
(9) Soakaway pits located near individual or communal septic systems should have a low potential for groundwater mounding such that there will not be any interference with the operation of the septic systems.
(10) Clear stone should be used as the storage media (50 mm). Well graded coarse sand is also acceptable.
(11) The seasonally high water table should be ≥ 1 m below the bottom of the soakaway pit.
(12) The bottom of the soakaway pit should be ≥ 1 m above bedrock.
(13) The pit should be lined with non-woven filter cloth to prevent the native soils from filling the void spaces in the clear stone.
(14) The roof leader should discharge into the trench via a perforated pipe. The perforated pipe should extend the full length of the pit and should be located near the top of the storage layer.

Sump Pumping of Foundation Drains

(1) The seasonally high water table should be > 1 m below the building foundation.
(2) The depth to bedrock should be > 1 m below the building foundation.
(3) If a soakaway pit is used it should be located a minimum of 4 m away from all building foundations
(4) If a soakaway pit is used, the bottom of the trench should be located a minimum of 1 m above the seasonally high water table.
(5) If the foundation drains are being discharged directly to the surface, the discharge point at the ground surface should be located a minimum of 2 m away from all building foundations
(6) If sump pumps discharge to the ground surface they should discharge a minimum of 0.5 m above ground to prevent blockage problems during the winter/spring period.

7.2 Stormwater Conveyance Controls

Perforated Pipe Systems

(1) Adequate pre-treatment (grassed swales, oil/grit separators, oversized catch-basins) should be provided.
(2) The soils should be suitable for active infiltration (≥ 15 mm/h percolation rate)
(3) The seasonally high water table should be ≥ 1 m below the bottom of the perforated pipe storage/backfill trench.
(4) The bottom of the perforated pipe storage/backfill trench should be ≥ 1 m above bedrock.
(5) Anti-seepage collars should be incorporated to prevent the migration of backfill/storage material and to promote vertical recharge.
(6) The slope of the perforated pipe system should be reasonable flat (0.5% - 2%).
(7) Non woven filter fabric should be implemented at the interface between the native soil and the pipe bedding to prevent the native material from clogging the void space in the storage media.
(8) The perforated pipe should not be wrapped with filter fabric (so that the fines in stormwater do not clog the perforated pipe).
(9) Clear stone (50 mm) or Granular A should be used as the perforated pipe bedding.
(10) The minimum and maximum storage volumes that should be provided in the perforated pipe bedding are the runoff volumes from a 5 mm and 15 mm storm respectively.
(11) Smooth-walled (interior) perforated pipes should be used.
(12) A minimum diameter of 200 mm perforated pipe should be used to facilitate maintenance
(13) A by-pass pipe or overflow pipe should be implemented for all perforated pipe systems which collect road runoff.
(14) A maintenance plan should be provided indicating what maintenance is required and how it will be performed. The plan should indicate whether the system will be seasonally operated.

Pervious Catch Basins

(1) Adequate pre-treatment (grassed swales, oversized sumps and goss traps) should be provided.
(2) The soils should be suitable for active infiltration (≥ 15 mm/h percolation rate)
(3) The seasonally high water table should be ≥ 1 m below the bottom of the trench storage material or sand filter (if implemented).
(4) The bottom of the trench should be ≥ 1 m above bedrock.
(5) The minimum and maximum storage volumes that should be provided in the catch-basin exfiltration storage trench are the runoff volumes from a 5 mm and 15 mm storm respectively.
(6) Non-woven filter fabric should be implemented at the interface between the native soil and the trench to prevent the native material from clogging the void space in the storage media.

(7) Clear stone (50 mm) should be used as the trench storage media.
(8) The infiltration area associated with the catch-basin should be designed with an inlet by-pass such that it performs exactly like a regular catch-basin.
(9) A maintenance plan should be provided indicating what maintenance is required and how it will be performed. The plan should also indicate whether the system will be seasonally operated.

Grassed Swales

(1) The swale should convey flow with a velocity of approximately 0.5 m/s during a 25 mm design event.
(2) The swale should be relatively flat (< 1%)
(3) The bottom width of the swale should be ≥ 0.75 m
(4) The grass in the swale should be allowed to grow higher than 75 mm
(5) The swale system should accommodate both the major system and minor system flow events.
(6) The erosion potential downstream of enhanced grassed swale check dams should be assessed.
(7) The side slopes of the swale should not be steeper than 2.5:1.
(8) Ditch and culvert servicing is acceptable as long as the swale lengths are greater than the driveway culvert lengths and the swale lengths are ≥ 5 metres.

7.3 End-of-Pipe Stormwater Management Facilities

Wet Ponds

(1) There should be a minimum contributing drainage area of 5 ha.
(2) The sizing should be based on the appropriate habitat criteria, level of imperviousness, erosion criteria, and flooding criteria. (Chapter 4 provides criteria for water quality control, Chapter 3 (Section 3.6) provides interim erosion control).
(3) The pond should have a minimum length to width ratio of 3:1.
(4) The permanent pool should be an average of 1 to 2 m deep with a maximum depth of 3 m.
(5) The extended detention depth for water quality or erosion should be 1 to 1.5 m.
(6) There should be a gradated planting strategy (five zones - aquatic to upland) prepared for the pond which addresses safety and nutrient removal. Native species should be used.
(7) The grading around the pond should promote safety (terracing). There should be a minimum slope of 5:1 at the edge of the normal permanent pool for a distance of 3 m into pond and 3 m up the slope.

(8) If an orifice is used as the outlet control in a perforated riser pipe, it should be ≥ 50 mm in diameter (≥ 75 mm if not protected by a riser structure) to prevent clogging.

(9) If a submerged inlet is proposed, upstream surcharging should be analysed, upstream deposition should be assessed, and the potential for re-suspension should be addressed.

(10) The inlet should be deep (> 1 m) to address re-suspension concerns

(11) The outlet design should minimize the potential for clogging and should provide a 24 hour detention for the design storage.

(12) There should be a maintenance access into the pond.

(13) A maintenance pipe (gravity drained) should be provided in the pond if feasible.

(14) A maintenance plan should be provided indicating what maintenance is required and how it will be performed. The plan should indicate the steps required to remove sediment from the pond.

(15) Sediment forebays should be lined below the permanent pool with open block/stone to facilitate sediment removal.

(16) Forebay berms should be within 300 mm of the permanent pool elevation.

(17) The forebay should not exceed 33% of the wet pond surface area

(18) All pipes (inlet, outlet, conveyance through a forebay berm) should be set ≥ 0.6 m above the bottom of the pond.

Extended Detention Dry Pond

(1) There should be a minimum contributing drainage area of 5 ha.

(2) The sizing of the pond should be based on the appropriate habitat criteria, level of imperviousness, erosion criteria, outlet control (batch or hydraulic), and flooding criteria. (Chapter 4 provides criteria for water quality control, Chapter 3 (Section 3.6) provides interim erosion control).

(3) The pond should have a minimum length to width ratio of 3:1.

(4) Typical extended detention storage depths range from 2 to 3 m although aggressive planting strategies may limit this to 1 - 1.5 m.

(5) A planting strategy should be prepared which addresses three zones (shoreline, floodfringe, and upland). Native species should be used.

(6) If an orifice is used as the outlet control in a perforated riser pipe, it should be ≥ 50 mm in diameter (≥ 75 mm if not protected by a riser structure) to prevent clogging.

(7) Grading of the pond side slopes should be 4:1 or flatter (average).

(8) The erosion potential at the inlet should be assessed and addressed.

(9) The backwater potential at the high water level should be assessed and addressed.

(10) The outlet should have a maintenance or by-pass valve in case the outlet becomes clogged.

(11) Quantity control in extended detention dry ponds is acceptable if the pond includes a permanently wet forebay.

(12) The forebay should not exceed 33% of the dry pond surface area

(13) The outlet design should minimize the potential for clogging and should provide a 24 hour detention for the design storage.

Wetlands

(1) There should be a minimum contributing drainage area of 5 ha.

(2) The sizing of the wetland should be based on the appropriate habitat criteria, level of imperviousness, erosion criteria, outlet control (batch or hydraulic), and flooding criteria. (Chapter 4 provides criteria for water quality control, Chapter 3 (Section 3.6) provides interim erosion control).

(3) The outlet design should minimize the potential for clogging and should provide a 24 hour detention for the design storage.

(4) The wetland should have a minimum length to width ratio of 3:1. The length to width ratio should be based on the low flow paths through the wetland.

(5) Low flow paths or braids should be created through the wetland

(6) If an orifice is used as the outlet control in a perforated riser pipe, it should be ≥ 50 mm in diameter (≥ 75 mm if not protected by a riser structure) to prevent clogging.

(7) The permanent pool depth for a wetland typically averages between 0.15 to 0.30 m. The depth of the permanent pool should fluctuate in the wetland to create growing conditions for a greater diversity of plantings. Deep areas should be limited to 25% of the wetland area.

(8) The extended detention storage depth should typically be ≤ 1 m although site specific planting strategies will dictate the allowable active storage depth

(9) A planting strategy should be prepared which addresses five zones (submergent, emergent, shoreline, floodfringe, and upland). Native species should be used.

(10) The grading around the pond should promote safety (terracing, maximum slope of 3:1). There should be a minimum slope of 5:1 at the edge of the normal permanent pool for a distance of 3 m into pond and 3 m up the slope.

(11) The erosion potential at the inlet should be assessed and addressed.

(12) The backwater potential at the high water level should be assessed and addressed.

(13) A forebay should be provided if possible to defer maintenance of the wetland area.

(14) Sediment forebays should be lined below the permanent pool with open block/stone to facilitate sediment removal.

(15) Forebay berms should be within 300 mm of the permanent pool elevation.

(16) There should be a maintenance access into the wetland or forebay.

(17) The forebay should not exceed 20% of the wetland area.

(18) Wetlands which provide quantity control should include a forebay.

(19) A maintenance plan should be provided indicating what maintenance is required and how it will be performed. The plan should indicate the steps required to remove sediment from the wetland.

Infiltration Trenches

(1) The contributing drainage area should be ≤ 2 ha

(2) There are restrictions on the applicability of trenches for commercial and industrial land uses. (generally not recommended - soakaway pits for rooftops would still be acceptable).
(3) The soils should be suitable for active infiltration (≥ 15 mm/h)
(4) The seasonally high water table depth should be ≥1 m below the bottom of the trench
(5) The depth of bedrock should be ≥ 1 m below the bottom of the trench
(6) The trench storage media should be 50 mm diameter clear stone.
(7) The trench should be sized to ensure a detention time less than 48 hours (24 recommended for design)
(8) The bottom area of the trench can be compared to that given by Equation 3.8.
(9) Perforated pipes in the trench should be spaced approximately 1.2 m apart
(10) The trench length should be maximized parallel to the direction of flow
(11) There should be a by-pass or overflow pipe designed into the trench system.
(12) Groundwater mounding should be assessed.
(13) Non-woven filter fabric be installed at the interface of the trench and the surrounding native material
(14) A surface trench should have a layer of non-woven filter cloth 150 - 300 mm below the ground surface in the stone storage media.
(15) A surface trench should have a 20 m wide filter strip pre-treatment SWMP
(16) The sand layer should be approximately 0.3 m thick. Peat can be used in the sand layer, however, fibric or hemic peat should be used.
(17) Aesthetic planting can be done over/around trenches. Plants with deep roots should be avoided since they could puncture the filter cloth surrounding the trench.
(18) Trenches should be constructed at the end of the development. Native species should be used.
(19) Trenches should not be designed specifically for water quantity control.
(20) A maintenance plan should be provided indicating what maintenance is required and how it will be performed.

Infiltration Basins

(1) The maximum contributing drainage area is 5 ha.
(2) There are restrictions on the applicability of infiltration basins for commercial and industrial land uses. (generally not recommended - basin control of rooftop drainage would still be acceptable).
(3) The soils should be suitable for surface infiltration (percolation rate ≥ 60 mm/h)
(4) The seasonally high water table should be ≥ 1 m below the bottom of the basin
(5) The depth to bedrock should be ≥ 1 m below the bottom of the basin
(6) The depth of active storage should be ≤ 0.6 m to prevent compaction from the weight of water
(7) The depth of storage should allow the entire storage volume to infiltrate with 24 to 48 hours (24 hours recommended for design).
(8) A by-pass path or pipe should be incorporated for maintenance and/or seasonal operation.

(9) A planting strategy can be developed and should be tailored to the depth of ponding in the basin. Deep rooted legumes will help increase porosity and maintain infiltration potential.
(10) Groundwater mounding should be assessed.
(11) Infiltration basins should be constructed at the end of development.
(12) Infiltration basins should not be designed specifically for water quantity control.
(13) A maintenance plan should be provided indicating what maintenance is required and how it will be performed.

Filter Strips

(1) There should be a maximum contributing drainage area of 2 ha.
(2) The filter strip should discharge into a buffer area or watercourse.
(3) A level spreader should be incorporated upstream of the filter strip
(4) The slope of the filter strip should be < 10%, and ideally < 5%
(5) The spreader, and hence width of the filter strip, should convey the peak flow from a 4 hour 10 mm storm with a depth of 50 to 100 mm.
(6) Storage should be provided upstream of the level spreader to detain the flows in excess of the spreader discharge for the 4 hour 10 mm storm.
(7) The length of the filter strip parallel to the direction of flow should be 10 m to 15 m for slopes of 1% to 5%; and 15 m to 20 m for slopes of 5% to 10%.
(8) Native species should be used for the vegetation in the filter strip.
(9) Filter strips should not be designed for erosion or quantity control.

Buffer Strips

(1) The watershed or subwatershed plan should provide guidance with respect to the necessary valley and stream corridor widths. In the absence of watershed/subwatershed planning Table 3.4 can be reviewed for appropriate valley and stream corridor buffer zones.
(2) The buffer strip should not be designed for erosion or quantity control.

Sand Filters

(1) The maximum contributing drainage area should be ≤ 5 ha
(2) Sand filters should have a surface ponding depth ≤ 1 m.
(3) The sand filter depth should be approximately 0.5 m.
(4) An overflow or by-pass conveyance system should be included in the sand filter design.
(5) A layer of clear stone (50 mm diameter) should be implemented beneath the sand layer to collect the effluent.

(6) Perforated drainage tiles can be placed in the stone layer to collect the enhanced water. The perforated pipes should be installed at a maximum spacing distance of 1.2 m
(7) Non-woven geotextile filter fabric should be placed between the sand layer and the stone layer to prevent the sand from being conveyed into the stone voids and drainage tile.
(8) A layer of peat can be included before the sand layer for additional treatment. Fibric or hemic peat should be used.
(9) Sand filters should not be designed specifically for water quantity control.
(10) A maintenance plan should be provided indicating what maintenance is required and how it will be performed.

Oil/Grit Separators (Three Chamber)

(1) Three chamber oil/grit separators should be implemented for drainage areas ≤ 2 ha.
(2) Three chamber oil/grit separators should be implemented for commercial and industrial land uses (ie. shopping centre parking lots, service stations) where the potential for spills is high (not for residential).
(3) Three chamber oil/grit separators should provide 30 m^3 of wet storage per impervious hectare of drainage area.
(4) Three chamber oil/grit separators should be located off-line (ie. accept low flows only (runoff from a 10 mm storm) with a high flow by-pass)
(5) The length to width ratio in the chamber should be ≥ 3:1
(6) The first chamber (grit chamber) should provide 50% to 70% of the total wet storage in the separator.
(7) A baffle near the entrance to the chamber will reduce flow velocities through the chamber.
(8) The area of openings in the wall between the first chamber and second chamber should be 5% to 8% of the total wall area.
(9) The oil chamber should have an inverted elbow pipe outlet which should be oversized (>450 mm) and extend approximately 300 mm into the permanent pool
(10) The elevation of the outlet in the discharge chamber should be low enough to prevent backwater effects.
(11) A maintenance plan should be provided indicating what maintenance is required and how it will be performed.
(12) Oil / grit separators are not recommended for quantity or erosion control.

Oil/Grit Separators (Manhole)

(1) Oil Grit separators are best used for industrial and commercial parking lots. (Not recommended for low density residential subdivisions unless they can be implemented with a cost savings compared to regular servicing)
(2) The contributing drainage area (100% imperviousness) for each separator should be ≤ 1 ha.

(3) Generally used in conjunction with other vegetative SWMPs (ie. not the only control for the site).
(4) Manhole separators should provide 15 m^3/ha of permanent pool (wet) storage for each impervious hectare of drainage area.
(5) Manhole separators should be located downstream of any parking lot flow restrictors. If possible, they should be located on collector sewer lines before the collector joins the main sewer line (unless a flow restriction is implemented downstream).
(6) A maintenance plan should be provided indicating what maintenance is required and how it will be performed.
(7) Oil / grit separators are not recommended for quantity or erosion control.

Flow Splitters/By-Passes

(1) The by-pass should be set at design water level if the splitter/by-pass is designed to operate once a certain volume of runoff has been captured.
(2) An assessment of the hydraulics should be undertaken to assess the real time operation of the flow splitter (flow into SWMP, by-pass flow, depth of water at splitter and SWMP).
(3) The inlet to the SWMP should be sized to convey the 4 hour 25 mm storm for storage type SWMPs, the 4 hour 15 mm storm for infiltration or filtration type end-of-pipe SWM facilities, and the 4 hour 10 mm storm for oil/grit separators and vegetated filter strips.

7.4 Stormwater Management Practices Integration

(1) Table 3.5 indicates the potential for different types of SWMPs to address various water management concerns.
(2) Infiltration targets from a subwatershed plan should be variably applied at a site level based on Equation 3.10.
(3) The assessment of end-of-pipe stormwater facility storage requirements should be adjusted for the lot level storage that is provided. The lot level storage should be adjusted based on a longevity factor. Longevity factors are provided in Table 3.7 based on the surrounding native material.
(4) Table 3.6 provides guidance for integrating stormwater storage assessments for stormwater lot level controls, stormwater conveyance controls, and end-of-pipe stormwater management facilities.

8.0 SWMP DESIGN EXAMPLES

8.1 Case I: Subwatershed Plan Governs Development

The proposed development is within an area which has a subwatershed plan. The subwatershed plan has identified the following stormwater management criteria:

i) quantity control to reduce the 1 in 5 year post-development peak flow to pre-development levels

ii) quality control to detain the runoff volume from a 25 mm rainfall event for 24 hours

ii) erosion control equivalent to 100 m^3/ha to be detained for 24 hours

iii) baseflow maintenance of 10 mm/ha based on soils with a percolation rate of 70 mm/h

The proposed site is 4.5 ha and will consist of 100 townhouses with a total imperviousness of 63%. The soils in the area have an average percolation rate of 50 mm/h.

Based on the subwatershed plan, the total developed area will require 450 m^3 for erosion control. Using OTTHYMO (Wisner and P'ng, 1983), the runoff volume was modelled for the total site for a 4 hour 25 mm rainfall event. Approximately, 566 m^3 is required for water quality control. Therefore, stormwater management controls are required to detain 566 m^3 for 24 hours to address water quality and erosion control criteria.

8.1.1 Lot Level Controls

Reduced Lot Grading

Based on the soils and the type of development, the lot grades will be reduced from 2% to 0.5%. Since the land is naturally flat, reduced lot grading will be feasible. The lots will be graded at 2% within 4 m of the building and at 0.5% for the remainder of the lot. Using Equation 3.11, the pervious depression storage was adjusted based on the longevity factor.

$DSP = 4.67 + (2 - G) f$ Equation 3.11 Pervious Depression Storage Adjustment

where : $G =$ 0.5% (lot grading)
$f =$ 0.75 (longevity factor)

Therefore, the DSP used in the OTTHYMO model was 5.8 mm to account for the reduced lot grading.

Roof Leader Discharge to Soakaway Pits

Since rooftop drainage is considered "clean water", the roof leaders from the buildings will be discharged to rear yard soakaway pits. The trenches will be located approximately 4 m away from the buildings and approximately 1.5 m above the seasonally high water table. They will be filled with 50 mm diameter clear stone. Each trench will be lined with non-woven filter cloth to prevent clogging of the stone. The appropriate bottom area of each trench was calculated using Equation 3.8. One soakaway pit will serve a block of 4 townhouse units. Therefore, each trench will store a maximum volume of 20 mm over the rooftop area of 4 units (approximately 400 m^2).

$$A = \frac{1000V}{Pn\Delta t}$$ Equation 3.8 Infiltration Trench Bottom Area

where:

$V =$ 8 m^3 (runoff volume to be infiltrated: 20 mm x 400 m^2 rooftop area for 4 units)
$P =$ 50 mm/h (percolation rate of surrounding native soil)
$n =$ 0.4 (porosity for clear stone)
$\Delta t =$ 24 h (retention time)

Therefore, the bottom area of the trench needs to be at least 16.7 m^2. Based on the lot configuration and open space areas, soakaway pits can be implemented which are 2 m wide and 8.5 m long. For the storage volume of 8 m^3, the pit needs to be 1.2 m deep. For the 100 units, there will be a total of 25 trenches. Based on Equation 3.1, the maximum allowable soakaway pit depth is as follows:

Maximum Allowable Soakaway pit depth = $P\,T$ Equation 3.1

where:

$P =$ 50 mm/h (minimum percolation rate)
$T =$ 24 h (drawdown time)

Equation 3.1 indicates that the maximum soakaway pit depth is 1.2 m. Therefore, the 1.2 m deep soakaway pit is within the maximum soakaway pit depth as calculated. The following equation was used to calculate a rating curve for input to P.C. OTTHYMO based on the storage and outflow for all the soakaway pits:

$$Q = f \times (P \div 3{,}600{,}000) \times (2LD + 2WD + LW) \times n$$ Equation 3.15 Soakaway Pit Rating Curve

$V = LWD \times n \times f$

where:

$f =$ 0.75 (longevity factor)
$P =$ 50 mm/h (native soil percolation rate)
$L =$ 212.5 m (total length of the soakaway pits)
$W =$ 2 m (width of each soakaway pit)
$D =$ 1.2 m (depth of water in the soakaway pit)
$n =$ 0.4 (void space in the soakaway pit clear stone)

Therefore, for a volume of 153 m^3, the discharge will be 0.004 m^3/s. This rating curve was modelled using P.C. OTTHYMO and the ROUTE RESERVOIR command for a 4 hour 25 mm storm to assess the contribution of the soakaway pit storage in the determination of end-of-pipe water quality storage requirements. Overflows from the trench storage were added to the runoff from the rest of the site.

8.1.2. Conveyance Controls

Pervious Pipe Systems

The townhouse development will be serviced with traditional curb and gutters. Groundwater contamination is not an issue for this development since a shallow aquifer feeds the stream and the road is local and will not be salted or sanded. Therefore, pervious pipes will be used with regular storm sewers for overflows. The municipality's standards allow pervious storm sewer systems. Grassed boulevards will be used as pre-treatment for the stormwater runoff. A total length of 260 m (130 m on each side of the roads) of perforated pipe with 50-12.7 mm diameter perforations per metre will be used. The 200 mm diameter perforated pipe will be set at 0.5% slope to promote exfiltration. Clear stone (50 mm) will be used for pipe bedding. The bedding will be surrounded with non-woven filter fabric to prevent the native soil from clogging the voids. The maximum depth will be 1.2 m as calculated previously using Equation 3.1. A typical pervious pipe design is shown in Figure 3.7.

Based on the following equation, a rating curve was estimated for the exfiltration flow as a percentage of the pipe flow.

$$Q_{exf} = (15A - 0.06\,S + 0.33)\,Q_{inf} \qquad \text{Equation 3.16 Perforated Pipe Exfiltration}$$

where:

$Q_{exf} =$ exfiltration flow through pipe perforations (see Table 8.1)
$f =$ 1.0 (longevity factor)
$A =$ 0.006 m^2/m (area of perforations/m length of pipe)

$S =$ 0.5% (slope of pipe)
$Q_{inf} =$ flow in pipe (see Table 8.1)

Table 8.1: Head Versus Exfiltration Flow for Perforated Pipe

Depth of water in pipe (m)	Flow in Pipe (m^3/s)	Exfiltration Flow (m^3/s)
0	0	0
0.025	0.001	0.0004
0.050	0.003	0.001
0.075	0.0065	0.003
0.100	0.0120	0.005
0.125	0.0165	0.007
0.150	0.021	0.008
0.175	0.022	0.009
0.200	0.023	0.009

The following equation was used to determine the amount of storage volume available within the clear stone pipe bedding.

$$V = LWD \times n \times f$$

where:

$L =$ 260 m (length of pervious pipe and stone)
$D =$ 1.2 m (depth of stone)
$W =$ 3.0 m (width of stone)
$n =$ 0.4 (void space for clear stone)
$f =$ 0.75 (longevity factor based on native soil)

Therefore, the actual available volume within the storage media is 281 m^3. The COMPUTE DUHYD command in P.C. OTTHYMO was used to divert the peak exfiltration flow to the pipe bedding (The DUHYD command was used since DIVERT HYD is not available in P.C. OTTHYMO. The DUHYD command splits flows based on a threshold flow rate whereas the DIVERT HYD command allows a rating curve for the division of flow. It is preferable to use DIVERT HYD, ie. OTTHYMO 89, wherever possible). The exfiltrated flow was routed

through the storage volume using the ROUTE RESERVOIR command. The outflow from the pipe bedding was calculated based on Equation 3.15.

$Q = f \times (P \div 3{,}600{,}000) \times (2LD + 2WD + LW) \times n$ Equation 3.15 Soakaway Pit Rating Curve

where:

f = 0.75 (longevity factor)
P = 50 mm/h (native soil percolation rate)
L = 260 m (total length of the soakaway pits)
W = 3.0 m (width of each soakaway pit)
D = 1.2 m (depth of water in the soakaway pit)
n = 0.4 (void space in the soakaway pit clear stone)

Therefore, the outflow from the pipe bedding is 0.006 m^3/s. All overflows were separated from the exfiltrated flows once the pipe bedding storage was exceeded. The overflows were conveyed in the regular storm sewer to determine end-of-pipe stormwater management requirements.

Based on the OTTHYMO output, the entire pipe bedding storage is not required. Therefore, as a cost saving measure, the storage volume was reduced to 140 m^3 (width was reduced to 1.5 m and the corresponding outflow was 0.004 m^3/s). **Note** : An alternative approach would have been to increase the number of perforations, and hence, the exfiltration in the perforated pipe.

8.1.3 End-of-Pipe SWMPs

Quality Control

According to the runoff volume reported in the OTTHYMO modelling, the required end-of-pipe storage is 275 m^3. The contributing drainage area and runoff volume are too small for the design of a wet pond or wetland. Therefore, a sand filter is recommended to provide the remaining water quality control. Based on the area available for the sand filter, Equation 3.18 was used to calculate the outflow from the sand filter.

$Q = f \times (P \div 3{,}600{,}000) \times (LW \times n)$ Equation 3.18 Sand Filter Discharge

where:

f = 1.0 (longevity factor based on the percolation rate for sand)
P = 210 mm/h (percolation rate for sand)
L = 32 m (length of the filter)
W = 8 m (width of the filter)
n = 0.25 (void space in the sand filter)

Therefore, the outflow from the filter will be 0.004 m^3/s. The storage available within the sand filter is 32 m^3. Storage to a depth of 1.0 m above the sand filter will be used to provide 256 m^3 of active storage. P.C. OTTHYMO and the ROUTE RESERVOIR command were used to model the storage and outflow rating curve.

Quantity Control

To provide control of the 1 in 5 year post-development peak flow, a dry pond is recommended which will receive 1 in 5 year flows from the storm sewers. The pond will provide 520 m^3 of storage at approximately 1.0 m depth. The outlet was sized to control the 1 in 5 year post-development peak flow to the pre-development flow.

8.1.4 Baseflow

The reported percolation rate of the soil is actually 50 mm/ha. Therefore, using Equation 3.10, the actual infiltration target is 7 mm/ha.

$$I = V \frac{P_{site}}{P_{SWP}}$$

Equation 3.10 Site Specific Infiltration Adjustment

where:

V	=	10 mm/ha (Target Volume of Infiltration from subwatershed plan based on a specific storm event)
P_{site}	=	50 mm/h (Percolation rate of site specific soils)
P_{SWP}	=	70 mm/h (Percolation rate of soils used in subwatershed plan)

Based on the infiltration measures recommended for this site, the total amount of recharge is 14.73 mm/ha which is greater than the required 7 mm/ha to meet the adjusted infiltration target.

8.1.5 Summary of Case I

Based on the stormwater management criteria outlined in the subwatershed plan for this site, quantity control, quality control, erosion control, and baseflow maintenance are required. The following stormwater management design will meet each of these criteria.

i) the 1 in 5 year post-development peak flow will be controlled with a dry pond approximately 520 m^3 in volume

ii) the reduced lot level grading and soakaway pits will reduce the required water quality storage by storing 15 mm (based on the longevity factor) of runoff from the roof area (approximately 150 m^3)

iii) the pervious pipe system will further reduce the water quality storage by providing storage in the pipe bedding (approximately 140 m^3)

iv) the sand filter will provide the remaining water quality storage (approximately 275 m^3)

v) the stormwater management controls will double the required baseflow contribution (approximately 14 mm/ha)

vi) the measures designed for water quality control will also provide erosion control benefits

8.2 Case II: No Subwatershed Plan Governs Development

In the absence of watershed/subwatershed planning, Chapter 4 of the SWMP manual was used to provide guidance on the design of stormwater management controls for a 50 ha subdivision. The proposed level of imperviousness for the site is 55%. The entire development will consist of 950 single detached housing units on typical 12 m x 30 m lots. Since there are no flood damage sites downstream of the site, and the site is located at the downstream end of the watershed, the site does not require flood control. The MNR habitat classification for the receiving water course is Type 2.

8.2.1 Lot Level Controls

Based on the soils, the potential for use of lot level controls is low. The soils have a percolation rate of 20 mm/h, and within this municipality, flat lot grading (< 2%) is not permitted. Also, the potential for contamination of the groundwater is a concern. Therefore, the only lot level control recommended for this site is soakaway pits for rooftop drainage.

Roof Leader Discharge to Soakaway Pits

Since rooftop drainage is considered "clean water", the roof leaders from the buildings will be discharged to rear yard soakaway pits. The trenches will be located approximately 4 m away from the buildings and approximately 1.5 m above the seasonally high water table. They will be filled with 50 mm diameter clear stone. Each trench will be lined with non-woven filter cloth to prevent clogging of the stone.

According to Table 3.7, the water quality storage requirements for the site should be reduced based on the use of soakaway pits. The appropriate bottom area of the trench was calculated

using Equation 3.8. Each rooftop is approximately 102 m^2. To store the maximum volume of 20 mm over the rooftop area, Equation 3.8 was used to calculate the bottom area required.

$$A = \frac{1000V}{Pn\Delta t} \qquad \text{Equation 3.8 Infiltration Trench Bottom Area}$$

where:

V = 2.04 m^3 (runoff volume to be infiltrated for 1 lot)
P = 20 mm/h (percolation rate of surrounding native soil)
n = 0.4 (porosity for clear stone)
Δt = 24 h (retention time)

Therefore, the bottom area of each trench would have to be 10.6 m^2. An area of 5.4 m^2 can be accommodated on each lot (1.2 m wide and 4.5 m long). Based on Equation 3.1, the maximum allowable soakaway pit depth is as follows:

$$\text{Maximum Allowable Soakaway pit depth} = P\,T \qquad \text{Equation 3.1}$$

where:

P = 20 mm/h (minimum percolation rate)
T = 24 h (drawdown time)

The maximum soakaway pit depth is 0.5 m. Based on the maximum depth and bottom area which can be accommodated, 10 mm of roof drainage can be accommodated in the soakaway pits.

A total of 1026 m^3 storage will be provided in soakaway pits for the subdivision (950 lots).

8.2.2 Conveyance Controls

Traditional curb and gutters will service this development. Based on the infiltration rates for the soils on this site and the potential for groundwater contamination, pervious pipes are not recommended.

8.2.3 End-of-Pipe SWMPs

A wet pond was chosen as the end-of-pipe stormwater management facility for this subdivision. According to Table 4.1, the design of a wet pond will require 110 m^3/ha of storage which corresponds to the following storage volumes for 50 ha: 3500 m^3 for permanent pool and 2000 m^3 for extended detention storage. The wet pond will be located outside of the floodplain and

will have a length to width ratio of 4:1. The permanent pool will be 2 m deep, and the extended detention storage will be approximately 1.25 m deep.

Storage Requirements

Equation 3.14 determines the reduction in the required end-of-pipe water quality storage volume (active storage) as given by Table 4.1, based on the use of soakaway pits for rooftop drainage.

WQS = ((TA - TRA) x Table 4.1 value)+((TRA x Table 4.1 value) - TPV) Equation 3.14

where:

WQS = Water quality storage (m^3)
TA = 50 ha (Total area of site)
TRA = 9.69 ha (Total roof area for all 950 lots)
Table 4.1 = 110 m^3/ha (Storage value from Table 4.1 for the lot)
TPV = 513 m^3 (adjusted soakaway pit storage)

and TPV = LWD x n x f

where:

f = 0.5 (longevity factor)
L = 4275 m (length of all soakaway pits)
W = 1.2 m (width of each soakaway pit)
D = 0.5 m (depth of each soakaway pit)
n = 0.4 (void space in the soakaway pit clear stone)

Therefore, the required end-of-pipe active water quality storage volume is reduced from 2000 m^3 to 1487 m^3.

Temperature

Since the receiving water course is sensitive to temperature changes, Equation 4.1 was used to calculate the temperature change in the stream. Equation 4.2 was used to calculate the average urban runoff temperature.

$$\Delta t = \frac{QT+q(Turb+\nabla t)}{(Q+q)} - T$$ Equation 4.1 Temperature Mass Balance

where :

Q =	0.233 m^3/s (average monthly summer daily maximum flow rate in the stream)
T =	20 °C (average monthly summer temperature in the stream)
Turb =	20.2 (average urban runoff summer temperature)
q =	0.03 m^3/s (average flow from SWMP during a 15 mm storm event)
∇t =	5.1 °C (average increase in temperature by SWMP type (Table 4.2))

Turb = 15.8 + 0.08 (55) Equation 4.2 Urban Runoff Temperature

Therefore, the change in stream temperature (Δt) is 0.60°C which is within the range of allowable changes as listed in Table 4.3.

Erosion

Since a subwatershed study has not been performed for this site, the runoff from the 25 mm rainfall event will be detained for 24 hours for erosion control. The required volume is 6875 m^3 which is greater than the 1487 m^3 required for water quality control (Table 4.1). The required volume for the pond will be decreased by the soakaway pit volume for a total required volume of 6362 m^3 (6875 m^3 - 513 m^3 provided by the soakaway pits).

The soils in the area are clayey silts and silty clays. Therefore, according to Figure 4.5, the critical velocity for a 0.01 mm size of particle is approximately 45 cm/s or 0.45 m/s. P.C. OTTHYMO was then used to model the erosion control volume to determine if the critical velocity is surpassed in the downstream channel. The uncontrolled post-development flows exceed the critical velocity resulting in an index value of 625.25 based on Equation 4.4.

$E_i = \Sigma (V_t - V_c) \Delta t$ Equation 4.4 Erosion Index

where:

E_i=	Erosion Index
V_t=	1.18 m/s (Velocity in the channel at time t=1.5 h (>V_c))
	0.72 m/s (Velocity in the channel at time t=1.667 h (>V_c))
	0.49 m/s (Velocity in the channel at time t=1.834 h (>V_c))
V_c=	0.45 m/s (Critical velocity above which erosion will occur)
Δt=	601.2 s (timestep (0.167 h))

Flows under pre-development and controlled post-development conditions do not exceed the critical velocity. Therefore, the 25 mm control is adequate for this site.

Drawdown Time

The drawdown time in the pond can be estimated using Equation 3.2.

$$t = \frac{2\ A_p}{CA_o(2g)^{0.5}} \quad (h_1^{0.5} - h_2^{0.5})$$ Equation 3.2 Drawdown time

or if a relationship between A_p and h is known (ie. $A = C_2h + C_3$)

$$t = \frac{0.66C_2\ h^{1.5} + 2C_3h^{0.5}}{2.75\ A_o}$$

where:

$A_p =$ varies (surface area of the pond)
$C =$ 0.62 (discharge coefficient)
$A_o =$ 0.04 m^2 (cross-sectional area of the orifice for 226 mm diameter)
$g =$ 9.81 m/s^2 (gravitational acceleration constant)
$h_1 =$ varies (starting water elevation above the orifice)
$h_2 =$ varies (ending water elevation above the orifice)
$h =$ 1.09 m (maximum water elevation above the centre-line of orifice)
$C_2 =$ 4571 (slope coefficient from the area-depth linear regression)
$C_3 =$ 3447 (intercept from the area-depth linear regression)

The linear regression was based on the area versus depth (y) listed.

for:

$A_p = 3136\ m^2$ $\quad h_1 = 0$ m
$A_p = 3969\ m^2$ $\quad h_1 = 0.25 - 0.11$ m
$A_p = 4900\ m^2$ $\quad h_1 = 0.5 - 0.11$ m
$A_p = 5929\ m^2$ $\quad h_1 = 0.75 - 0.11$ m
$A_p = 7056\ m^2$ $\quad h_1 = 1 - 0.11$ m
$A_p = 8036\ m^2$ $\quad h_1 = 1.2 - 0.11$ m

$$A_p = 4571\ h + 3220$$

$$A_o = \frac{3017 + 6894}{2.75\ t}$$

$$t = \frac{3865}{A_o}$$

Therefore, the drawdown time in the pond is equal to 96643 s or 26.8 hours.

Forebay Length

The forebay size depends on several calculations.

1. Settling Calculations

The first step is to determine the distance to settle out a certain size of sediment in the forebay. The settling velocities for different sized particles can be estimated from the stormwater particle size distribution monitoring data by the U.S. EPA listed in Table 3.3. Equation 3.3 defines the appropriate forebay length for a given settling velocity.

$$Dist=\sqrt{\frac{rQ_p}{V_s}}$$ Equation 3.3 Forebay Settling Length

where:

r = 2:1 (Length to width ratio of forebay)
Q_p = 0.1 m^3/s (Peak flowrate from the pond during design quality storm)
V_s = 0.0003 m/s (Settling velocity for 0.15 mm diameter particles)

Therefore, the forebay should be 26 m long to settle particles approximately 0.15 mm diameter in size.

2. Dispersion Length

Equation 3.4 provides a simple guideline for the length of dispersion required to dissipate flows from the inlet pipe. It is recommended that the forebay length is such that a fluid jet will disperse to a velocity ≤ 0.5 metre/second at the forebay berm. The fluid jet should be based on the capacity of inflow pipe (if the pipe is ≤ 10 year pipe). In this subdivision the pipe will be designed to convey the 5 year storm flows. A flow splitter will not be implemented.

$$Dist = \frac{8Q}{dV_f}$$ Equation 3.4 Dispersion Length

where:

$Q =$ 5.1 m^3/s (inlet flowrate)
$d =$ 2 m (depth of the permanent pool in the forebay)
$V_f =$ 0.5 m/s (desired velocity in the forebay)

Therefore, the forebay length should be 40.8 m for the peak flow during a 5 year storm.

A guideline for the minimum bottom width of this deep zone is given by :

$$\text{Width} = \frac{\text{Dist}}{8}$$

Equation 3.5 Minimum Forebay Bottom Width

Therefore, the forebay deep zone should be at least 5.1 m wide.

Therefore, the forebay will be 45 m long and 20 m wide (based on an approximate 2:1 length to width ratio). The velocity of the flow as it moves through the forebay will be as follows:

$$\text{Velocity} = \frac{Q}{A}$$

where:

$Q =$ 5.1 m^3/s
$A =$ 22 m^2 (cross-sectional area)

Therefore, the average velocity through the forebay will be 0.23 m/s. This velocity is acceptable since it is less than the 0.45 m/s permissible velocity to prevent erosion as noted previously.

Given the results of Equations 3.3 and 3.4, the forebay length will be 45 m long and 20 m wide. The permanent pool volume of the forebay will be approximately 900 m^3.

3. Cleanout Frequency

Based on Table 5.3, the annual sediment loading for this site will be approximately 2300 kg/ha or 1.9 m^3/ha. Therefore, based on the volume of the forebay (900 m^3) and a the pond removal efficiency of 70% (Level 2 Protection), the forebay will be required to be cleaned out every 13.5 years. This is acceptable to the municipality since it is greater than the 10 year minimum cleanout frequency.

Forebay Berm

The forebay will be separated from the rest of the pond by an earthen berm. The berm will be submerged slightly below the permanent pool. Low flow pipes will be installed in the berm to convey low flows from the forebay to the pond. The conveyance pipes will be installed in the berm at 0.6 m above the bottom of the forebay. A maintenance pipe will also be installed in the berm to drawdown the forebay for maintenance purposes.

8.2.4 Summary of Case II

According to Table 4.1, a wet pond for this site will require 3500 m^3 for a permanent pool and 2000 m^3 for active storage to provide water quality control. For erosion control, the required volume is 6875 m^3 based on the 25 mm rainfall event. The following SWMPs have been designed to meet these criteria:

i) Soakaway pits will accommodate 10 mm of runoff from the roof area which will reduce the required end-of-pipe active storage requirements by 513 m^3.

ii) A wet pond will provide the end-of-pipe stormwater management (water quality and erosion) control. The pond will provide 3500 m^3 of permanent pool storage and 6362 m^3 of active storage.

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APPENDIX A

WATERSHED / SUBWATERSHED PLANNING

A.1 Watershed/Subwatershed Planning and the Implications of Scale

Watershed and subwatershed plans and the process used to develop them are strategic in nature. The objective of the overall process is to develop an understanding of the biological resources and physical systems, their inter-dependencies and linkages. This understanding is developed with the participation and assistance of regulatory agencies, municipalities and the public so that "team building" takes place during the process. The importance of this aspect of the process should not be minimized. A strong understanding of the watershed or subwatershed and its priorities will greatly reduce the potential for conflicts between the mandates of different agencies and will facilitate the final permitting process.

The watershed/subwatershed planning process is required to provide:

- Input to municipal land use plans which will ensure protection of the important natural systems and resources in a manner acceptable to regulatory agencies.
- Direction to proponents of development regarding the level and types of control or management actions required.
- Direction to agencies and municipalities regarding protection and preservation of valley systems, wetlands and green space.
- Direction to agencies and municipalities regarding remediation, rehabilitation/enhancement programs, and pollution prevention.
- Direction to agencies and municipalities regarding management programs (fisheries, agriculture, public education etc.)
- An implementation strategy.
- A monitory strategy.

The level of detail provided under each of these will depend the scale of plan being prepared. At the highest level (large watershed plan) the direction provided will be at a policy level. Input to land use planning will include policy requirements, and specification of subwatershed planning areas.

At the most detailed subwatershed plan level the direction provided will be more precise and specify such things as required buffers, SWMP type, features and design requirements, and specific stream reaches to be rehabilitated or enhanced. In some instances, where an important natural function could be impaired the subwatershed plan may specify maximum levels of imperviousness, recommend development forms or otherwise restrict the allowable area of development.

The size of the watershed will generally have a decisive effect on how the watershed/subwatershed planning process is organized and staged. Watershed scale influences the amount and type of data and information which can be practically and economically collected in one study, the detail of land use planning information which is available or which can be practically incorporated into the plan, the assessment and modelling techniques which can be usefully employed and the level of public consultation which is possible. The quality and detail of information will in turn influence the type and detail of management decisions and direction that can be provided. While policy level management decisions and first level strategic planning can often be completed with previously published data on biological and physical systems, official plan level land use information, simplified assessment and modelling techniques and regional scale public consultation, such information is adequate to make the decisions required in a subwatershed plan. Subwatershed plans are expected to be explicit in their direction for protection, rehabilitation and enhancement, in particular areas and locations. This necessitates much more detailed information so that defensible decisions can be made.

In recent years, experience has been gained with the watershed/subwatershed planning process. Different approaches have been employed which reflect the scale of the watershed, data considerations and growth pressures. Three examples of approaches that have been used are provided below:

Examples of Watershed/Subwatershed Planning Approaches

1. Large Watershed

- widespread, dispersed development pressure.
- issues unrelated to urban development may be of importance.
- naturally sustainable subwatersheds exist within the larger unit.
- level of planning information is highly variable.
- development will proceed over an extended period (decades).
- relatively large basin.
- watershed plan usually initiated by a conservation authority.

Watershed Plan :
- done first, lays out strategy and policy; defines regional biological and physical systems and future subwatershed planning areas.

Subwatershed Plans :
- completed in advance of development but timing is based on growth. Defines details: boundaries of corridors, erosion protection locations, SWMP locations and capacities.
- required to meet specifications outlined in the

Watershed Plan.

- subwatershed plan is usually initiated by the local municipality, often in cooperation with the conservation authority.

Stormwater Management : Plans
- detailed design, servicing, integration. Stormwater management plan may be required on a sub-basin basis if planning information is insufficient at the subwatershed level.

If a watershed plan has not been done, subwatershed plans can proceed if:

1) conducted on the basis of a naturally sustainable unit;
2) the effects on and linkages to the larger watershed are addressed.

The subwatershed planning approach is listed under #3 below.

2. Small Watershed Approach

- development pressure is concentrated but does not cover the entire area.
- majority of tributaries are not viable or naturally sustaining if taken from the whole.
- drainage area is relatively small.
- planning information or development phasing will not allow completion of a comprehensive subwatershed plan in the majority of areas.

Watershed Plan : done first as in Example 1 above, but carries the technical studies (biological resources, groundwater, surface water) to a virtual end point. Defines sub-basins and specific issues for the future subwatershed plans.

Subwatershed Plan : detailed refinement only → number of facilities, locations, servicing, specific boundaries for protected areas and, if necessary, the development form.

Stormwater Management: Plan — detailed design, final servicing, integration with community.

3. Subwatershed Plan Approach

- development pressure is concentrated within a drainage area which is a naturally sustainable unit.
- a watershed plan has been completed or the drainage area is sufficiently small in relation to the overall watershed that impacts will be minor if development proceeds with local environmental sensitivity.
- planning and probable development phasing suggest that much of the sub-basin would be developed within the foreseeable future.

Subwatershed Plan :	completes work required of a watershed plan first then moves into detail for potential development areas. Subwatershed plan explicitly required to consider cross-boundary, regional linkages. Details provided for setbacks, buffers, SWMP types locations and sizes. Plan may specify need for sub-basin plans where planning information is insufficient.
Stormwater Management: Plan	detailed design, final servicing, integration with community. Stormwater management plan may be required on a sub-basin basis if planning information is insufficient at the subwatershed level.

Additional variations on these approaches are possible. The important thing to recognize is that the overall process remains the same, regardless of the approach adopted.

The key to successful water management planning lies in ensuring that an ecologically sound basis of planning is followed from the beginning of the watershed plan through to the end of the stormwater management plan. Where each planning component begins and ends may be determined on an individual basis. The determination is the responsibility of the multi-agency steering committee, and must rely on their professional judgement and knowledge of the subject area. The hierarchy of plans and the scope assigned to each will however ensure that:

- The first plan completed will be completed on the basis of a naturally sustainable unit (usually a watershed or large subwatershed). The boundaries will be defined based upon existing knowledge of biological and physical systems.
- Each subsequent planning level draws direction from the decisions made at higher planning levels and provides direction to subsequent planning levels.
- All decisions which relate to the levels protection needed or constraints on the use of land must be made prior to commencement of stormwater management plans.

A.2 Subwatershed Planning - Example of Early Projects Stages

A.2.0 Background Review

The first step in virtually all subwatershed planning studies is the background review. In many other types of study the background review is primarily a familiarization exercise conducted in order to provide a context for future work. This is not the case in a watershed or subwatershed plan. The background review is a major undertaking involving not just collection and review of information but also synthesis and interpretation between disciplines, evaluation of information against goals and objectives, identification of watershed problems and issues, identification of critical data deficiencies and development of a refined work plan. The background review should establish a good preliminary understanding of the major functions and linkages of the watershed or subwatershed. It should allow refinement and prioritization of goals and objectives. It should result in an directed work plan aimed at specific issues and weaknesses in the understanding of key watershed processes.

A.2.1 Information Collection

The collection of information in watershed and subwatershed planning studies should be undertaken with due regard to the scope of plan and the level of decisions and direction which will be expected from it. For watershed plans the information collected will be contained predominantly in regional scale or district publications for many of the disciplines. For some disciplines (e.g. water quality) however a lack of regional scale data may be expected in many cases and it will be necessary to seek data from site specific sources.

For subwatershed plans the first source of useful information will be the watershed plan, if one has been completed. Generally however a finer level of detail will be pursued during the background review for a subwatershed plan. Local scale reports or data will often be available from previous and pending development applications, and servicing studies, class environmental assessments and remedial action investigations. Care should be taken in the pursuit of detailed background information, as it is a task which can expand rapidly. Continual consideration of the need for the information in relation to goals and objectives being addressed is important. In some cases, the collection of information will be limited to cataloguing its source and availability pending a decision on the need for the information at the end of the background review. The watershed plan if it exists can be useful in directing the collection of detailed information. If a particular resource type was found to be of little importance in the strategy developed in the watershed plan, exhaustive efforts in pursuit of detailed information on that resource will not likely be warranted.

The Province's draft Interim Guide on Subwatershed Planning contains a section on information needs and identifies the types of information and potential sources. While this guidance is

provided in the draft subwatershed planning guideline, it applies equally to watershed and subwatershed plans.

Mapped, printed and computerized data will form the bulk of the information collection in the background review. There will also be the need for preliminary field investigations however. These are necessary to verify information, investigate anomalous reports, and to gain a general familiarity with the watershed or subwatershed. The preliminary field investigations are of particular use to disciplines which typically find a shortage of documented data (e.g. biologists, geomorphologists and water quality specialists). Where possible these preliminary field investigations should be conducted as a team so that the visual inspection stimulates interaction between the different disciplines.

The approach to preliminary investigations will be dictated by scale of the watershed under investigation. For large watersheds and subwatersheds, investigations will normally be limited to easily accessed points (such as road crossings of the stream) and areas of documented importance (boundaries of ESA, significant wetlands, major erosion sites etc.). At the smaller subwatershed scale it may be possible to walk selected portions of the stream and valley system.

The preliminary field investigations should include the taking of photographs in addition to field notes. These photographs when mounted and displayed at public consultation meetings, often stimulate discussions which reveal useful qualitative information and/or potential sources of information.

A.2.2 Synthesis of Information

The synthesis and integration of information is normally a multi-step process. The initial stage is the review, cataloguing and mapping of information and data deficiencies for each discipline or combinations of disciplines. The intent here is to build the picture of the watershed's key features and workings on a discipline by discipline basis based on existing and historical conditions. While the groupings of disciplines may vary according to the skills of the team, the following groupings are often used:

- Geology/Hydrogeology
- Hydrology/Hydraulics
- Geomorphology/Erosion
- Water Quality
- Aquatic Resources
- Terrestrial Resources
- Land use/Servicing

Once the background picture of each discipline is complete and conclusions drawn regarding the

key functions of each, then cross-discipline integration can begin. Cross-discipline integration can be promoted in a number of ways but internal team meetings involving team leaders for each discipline has been found to be useful. The number and length of team meetings will depend on the scale and complexity of information to be understood. Overall however, the agenda of the meeting(s) should include:

- Summary presentation by each discipline noting important characteristics and functions identified.

- Review of Goals and Objectives by the team (to include identification of limitations to goals imposed by physical systems).

- Review of existing problems or conditions and activities causing problems (existing sewer systems, land use, channelization, flood obstructions etc.)

- Review of Potential impacts of future land use change on key areas of concern (eg. groundwater flow and quality, flooding, erosion,. water quality, fisheries habitat, terrestrial and wetland resources).

- Review of legislative and policy framework (federal, provincial and municipal) in relation to goals, problems and potential impacts, in order to identify basic preservation and protection which will be required (eg. flood plain designation, hazard lands etc.)

- Review of potential management options to be considered.

- Review of potential assessment techniques to be utilized.

- Summarize primary data deficiencies by discipline.

Following the meeting or meetings on integration of the background information, the next step is the preparation of a series of issues and problem statements based on the understanding gained through the meeting(s). The purpose of this step is to focus the remainder of the study. The preparation of issue and problems statements should be done by either the project manager and a small group of senior staff, rather than by the specialists who assembled the background review information. This approach forces the synthesis of information and the identification of key systems. It should be noted that the issue and problem statements are not focused solely on the negative but rather must reflect the goals and knowledge of the watershed resources. In particular, where enhancement is cited as part of a particular goal, issues will arise as to how to accomplish this and whether proposed efforts will successfully lead to the desired enhancement. The issue and problem statements are, of course, circulated to individual discipline specialists for comment after they have been drafted. Several iterations may be

required before the final set of statements is agreed to by the entire team.

The form, the emphasis and the degree of inter-connection between the different issues and problem statements will vary markedly depending on the plan being prepared. If the plan is an upper tier plan, at either a watershed or large subwatershed scale (eg. no other plans have been completed) the statements will normally be strongly oriented to the preservation, protection and enhancement of key physical systems and natural resources of the planning area. If the plan is being prepared for a subwatershed following completion of an upper level plan, the issues and problem statements will be more oriented towards the how and where to implement management actions specified in the upper tier plan.

The Issue and Problem statements form the core around which a detailed workplan is formed. Different programs of data collection and development of assessment tools will be needed to address the statements. The statements however are also the key to communication with the Steering Committee and the public. As such, they must demonstrate the inter-connectivity of the watershed systems and resources and identify the reasons and need for specific information collection or assessment techniques.

A.2.3 Data Collection Needs

Collection of additional data and the development of highly sophisticated assessment tools can be an expensive undertaking. There is therefore a need to critically examine the necessity of filling identified gaps in the data base. It should be recognized that there will be a range of reasons for collecting data, that the cost of collecting different types of information will vary greatly and that the critical nature of particular types of data will depend on the significance of particular physical and biological systems and their inter-dependencies. Finally, there should be consideration of the types of management actions which may be implemented.

A.2.4 Consultation

The final steps in the background review phase are consultation with the Steering Committee and the public and, based on the input received, the preparation of a final detailed work plan. Consultation should include:

- Overview of the watershed.
- Review and refinement of the study goals and objectives.
- Preliminary conclusions regarding the importance of key systems, processes and resources.
- Discussion of potential opportunities and impacts (based on the issue and problem statements).

- Recommendations for the technical studies needed to facilitate preparation of the management plan.

The consultation should be oriented towards interaction and input rather than simply the provision of information. The consultation process should be viewed as the final step in the collection of background information. The study team will have developed a view of which systems are most important and what additional data is needed but this will have been developed based primarily on their technical understanding. Agency and public attitudes will provide an additional context which must be accounted for.

The consultation process will result in a final definition of what is to be accomplished by the plan and a final focusing of efforts to be undertaken. It should be recognized that there may be modification or prioritization of the goals and objectives, which may affect the data collection needs.

A.2.5 The Detailed Work Plan

The detailed work plan is the culmination of the background review phase. It specifies the purpose of the different technical studies to be undertaken, the techniques to be employed, and the costs and schedules for implementation. Responsibility for approval of the detailed work plan rests with the Steering Committee. The Committee must satisfy itself of that the work plan reflects their needs and the needs expressed by the public.

Budgetary questions will inevitably arise at this stage as there will be a need for reallocation of resources, allocation of contingency funds and in some instances increases in the budget available. Strong justification is of course required for increases in allotted budgets and there are always alternatives to be considered (scaling back or curtailing areas of investigation). It is the consultant's responsibility to clearly indicate the implications of such decisions and the limitations they may produce in the final decision making process and plan. The Steering Committee must consider these limitations on two levels. First, the implications to the level of direction that will be possible in the final plan should be considered. Second, the defensibility of the decisions taken in formulating the plan should be considered. Watershed and subwatershed planning is not a process which is currently subject to appeal and adjudication. The decisions made in the plan may be challenged before the Ontario Municipal Board however, once the plan bas been used as the basis for land use planning decisions. Care must be taken to ensure that strict adherence to a predetermined budget does not result in a weakening of the basis for the plan so that it fails to serve its purpose.

A.3 Subwatershed Planning - Examples of Technical Studies

The technical studies of a watershed plan include supplementary field studies, data collection and the development of assessment tools necessary to establish an acceptable understanding of the watershed and to make decisions regarding management options. The level of effort given to these activities is dependent on the nature of the watershed and the scope of the plan. As indicated previously, the technical studies to be undertaken are specified in the detailed work plan, completed at the end of the background review phase. Further discussion on the types of studies and options for their implementation are discussed in the following sections.

A.3.1 Basis for Technical Studies

There are normally four possible reasons for the conduct of technical studies, although work completed for one reason usually has secondary uses in the ultimate formulation of the watershed or subwatershed plan. The reasons for technical studies include:

- Establishing baseline information and the basis for long term monitoring to gauge success of the plan.
- Completion or upgrading of resource inventory information.
- Developing the tools necessary to evaluate management options.
- Refining or clarifying key inter-relations which govern the function of the watershed and its resources.

Baseline Information

Baseline information is usually collected for water quality, flow conditions and erosion monitoring, and in some cases for the aquatic community and wildlife community.

Examples of baseline studies include:

Flood Conditions :	Continuous flow monitoring at selected stations.
Water Quality :	Quarterly sampling of water chemistry at selected locations.
Erosion :	Surveying selected stream cross sections for future monitoring of erosion.
Aquatic Community	Fish collection to establish diversity and bio-mass levels; Herptile inventories to establish diversity; Benthic surveys to establish diversity.
Wildlife :	Inventories and/or live trapping to establish species present.

While each of these type of studies will produce useful information in developing an understanding of the watershed, their primary importance is as the beginning of a monitoring

program which will be continued into the future, as development proceeds. This is an important role in the watershed/subwatershed planning process. It should be recognized however, that much of the information collected will not normally be critical to the selection of management options. This is due to the fact that the data will be basically a "snapshot" in time, and cause/effect relationships linking the data collected to potential management options are largely qualitative or empirical in nature.

Inventories

Inventories are often conducted for vegetative communities, wildlife, aquatic communities. The level of effort involved in inventories can span a tremendous range. Rigorous inventory techniques conducted according to prescribed guidelines can be very time consuming and expensive. While such inventories are included in some watershed/subwatershed plans, this level of investigation is not normally warranted on the basis of the management decisions which will be made. There will be exceptions to this, especially where a locally valued natural area has been designated as an environmentally sensitive area, sensitive stream, or sensitive wetland. In general, however less rigorous inventory and mapping techniques are preferred in watershed plans where management techniques will focus on protection of important blocks, prevention of encroachment through buffers, and preservation of physical systems such as the groundwater regime.

Assessment Tools

Technical studies are normally required to formulate and calibrate models and other assessment techniques for use in the evaluation of options. While the models used will normally be in the public domain they must be formulated and calibrated for the particular watershed. In some instances, the models must be modified or expanded to allow simulation of specific items of importance.

Refining Understanding

The last but most important reason for conducting technical studies is to refine or increase the understanding of the key processes and functions which govern the watershed. These studies may include geological and hydrogeological field studies to define ground water movement, fish habitat surveys to determine limiting factors to the fishery, specific water quality studies directed at important fisheries related parameters such as temperature, pH or dissolved oxygen, water quality studies directed towards public health (eg. bacteria) or aesthetics (nutrients, turbidity), and continuous flow monitoring for baseflow characterization.

A.3.2 Field Studies

The number of possible field studies and techniques which may be used to collect important information is quite large and an exhaustive listing is not provided here. Individual specialists working within the context of a particular watershed are best suited to select the most cost effective techniques. A listing of some of the most commonly employed field studies and techniques is provided in the following sections.

<u>Surface Water System</u>

The studies most often undertaken for the surface water system include flow monitoring, stream cross section and profile surveys, erosion monitoring and water quality monitoring.

Flow Monitoring

Flow monitoring is one of the most commonly required forms of field survey because permanent flow gauges rarely provide the desired coverage for tributaries or the major ecological zones (eg. headwaters, middle reaches, etc.) of the watershed. On major watersheds the network of permanent flow gauges will provide the majority of information needed for main branch stations, but there may still be the need for monitoring of tributaries.

The type of plan being prepared and the scale of the watershed, influence the form which the flow monitoring takes. For large watersheds with a number of permanent gauges, the monitoring is normally directed towards the mouths of ungauged tributaries or subwatersheds. For smaller, largely ungauged watersheds the selection of monitoring stations is usually oriented towards locations on the main branch which represent charges in the ecological/geomorphological zones, or changes in existing or future land use (eg. a gauge is often placed at the upper extent of existing development). For subwatershed plans, the selection of flow monitoring sites is highly variable. Usually the selection of monitoring locations is based on knowledge of the fisheries potential and variations in land use. Where the subwatershed plan is being prepared subsequent to a watershed plan and is focusing primarily of details such as the location of facilities and definition of areas of constraint, flow monitoring may not be needed.

In many watershed and subwatershed plans the requirement for flow monitoring is specified in the terms of reference of the study. The most common practice is to require monitoring for a period of one year, or in some cases, for the duration of the study. If this is not the case, then locations and duration of flow monitoring will be specified in the detailed work plan. In either case, it should be recognized that the monitoring will be conducted over a relatively short interval (although gauging may continue after completion of the plan as part of an overall monitoring strategy). As such, the likelihood of occurrence of an major rainfall event will be small. The monitoring network established is therefore primarily of use in establishing the day

to day flow regime and the tributary's response to typical rainfall events that occur relatively frequently. As a result, although a year of monitoring may be carried out, sufficient data is often available after a few months, to allow characterization of the hydrologic response. In some instances, recognition of this is incorporated into the study design and some equipment is relocated from one area to another every four to eight weeks.

A variety of monitoring equipment is available for use. Success with continuous recording water level monitors has been noted. The equipment is battery operated and data can be downloaded in the field using portable computers. The equipment normally allows the recording of temperature as well as water level and it is often possible to add specific probes to measure other parameters such as dissolved oxygen and temperature. The equipment used may be consultant-owned or rented from suppliers. In many cases consideration should be given to purchase of the equipment by the client because the rental charges will approach or exceed the cost of purchase. If the monitoring is to be continued after the completion of the study, purchase is almost always the preferred option.

Flow monitoring is often supplemented through the collection of other related data. This may involve the use of rain gauges, evaporation pans, minimum/maximum thermometers, staff gauges and crest gauges. The use of this supplementary equipment is dictated by the nature of the watershed/subwatershed and the information needs.

Stream Cross Section and Profile Surveys

Available information on stream channel cross section and stream profile is often restricted to data which has been collected as part of flood studies. In many cases there is poor definition of the normal or low flow channel and especially in areas of limited relief (eg. wetlands), the actual location and extent of the water course may be in doubt. In such cases geodetic field surveys are often conducted. This is most commonly required for detailed watershed or subwatershed plans, rather than for larger scale watershed plans.

Erosion Monitoring

Streambank erosion is one of the more difficult areas of assessment in watershed and subwatershed plans. Our technical capability to predict the impacts of future land use change on streambank erosion is not precise, although it is well known that changing land use accelerates erosion. Generally, erosion management takes the form of a combination of preventive management (eg. runoff quantity control) and reactive correction.

A common approach employed on small watershed or subwatersheds is to establish a series of erosion monitoring stations. The location of stations is usually determined by a team of specialists with expertise in geology, erosion, geomorphology and fisheries habitat. Each station is surveyed and tied in to permanent markers so that it can be re-surveyed at intervals, as land

use change proceeds. At the time of the survey, samples are collected of the bed and bank material and an in-situ assessment of erosion potential is conducted. This information is used in conjunction with hydrologic and hydraulic modelling results during the evaluation and assessment stage of the plan to make decisions regarding the level of quantity control needed, for erosion protection.

Water Quality

Water quality programs are usually composed of a hierarchy of studies. The design of the water quality program should be tailored closely to the uses to be protected (eg. fisheries, public health), the current and future land use, expected problems (eg. pollution sources) and future monitoring requirements. In particular, the range of chemical parameters to be tested for should be reviewed within the context of the assessment techniques available and the management decisions which can be made. Collection of expensive water quality data which serves a very limited purpose is one of the most common difficulties in watershed and subwatershed studies.

The types of water quality studies often required include baseline sampling, sediment sampling, oxygen and temperature surveys and investigative sampling.

Baseline sampling normally involves the collection of water column samples on a quarterly basis for a minimum of a year. It serves several purposes:

- establishes basis for future monitoring of the combined effect of management actions taken as part of the plan.
- elicits differences in background water quality on different sub-tributaries with different land uses, geology or hydrogeology.
- determines factors which could limit watershed uses (eg. fisheries, public health).

Typically sampling consists of the collection of grab samples during normal (non rainfall) conditions. Sampling during the spring period may be timed to coincide with the melt. The parameter groups most often sampled are the conventional parameters (to include suspended and total solids, hardness, pH, conductivity, plus a range of chemical tests), nutrients and heavy metals. Industrial organics and herbicides/pesticides should only be included in sampling where there is a clear reason (eg. a known or suspected source). Inclusion of these latter parameters greatly increases the cost of lab analysis and they will rarely be found above detectable limits in the conditions encountered during baseflow sampling. It is preferable to direct resources to increasing the number of stations sampled rather than increasing the parameters sampled to include the more exotic contaminants.

Most labs which may be contracted to carry out the analysis of samples offer packages which cover the different groups of parameters. The package selected should be designed for surface water assessment and should have detection limits which are at or below provincial water quality

objective (PWQO) levels. Many commonly used packages were not designed for surface water assessment and the detection limits are too high to be of use in baseline sampling.

Sediment sampling at selected location is often conducted to supplement baseline sampling. Sediment samples integrate the accumulation and transference of contaminants and tend to concentrate them. If monitoring for organic chemicals is desired, it often best to begin with sediment sampling.

Temperature and dissolved oxygen surveys are often carried out in support of fisheries assessments or evaluation of nutrient problems. Temperature is normally measured whenever other sampling or field surveys are completed. In addition, the continuous temperature monitoring and the use of maximum/minimum thermometers is often integrated with the flow monitoring studies. Temperature data collected from a spatially distributed set of stations can provide useful information regarding areas of groundwater discharge.

Dissolved oxygen surveys are often carried out to assess limits to fishery potential or to assess the impact of algae blooms. Two methods, other than spot measurements are usually employed. First, oxygen probes can be coupled with flow monitoring devices and used to measure oxygen variation continuously. A more labour intensive technique is to conduct 24 hour dissolved oxygen surveys at selected locations during critical times of year.

The final category of water quality study is the investigative study. Such studies are normally aimed at specific problems or potential pollution sources. The impacts of landfill sites, storm water ponds, water pollution control plants, storm sewer outfalls or combined sewer overflows may be examined by combinations of sediment sampling and water column samples (upstream and downstream). Such studies are normally carried out at the subwatershed plan level and are limited to the level necessary to suggest the need for future pollution abatement studies as part of the ongoing management of the watershed.

Ground Water System

The types of studies which may be conducted in relation to the ground water system range from surficial investigations which can be conducted simply and with limited resources to very complex investigations at depth which may require a major commitment of resources. The level of investigation needed depends on the importance of the particular area to baseflow, fisheries, or water supply, the complexity of the geologic stratigraphy and hydrogeologic regime, and whether management techniques involving structural infiltration of runoff are to be employed.

Surficial Methods

A variety of inexpensive techniques may be used to supplement and confirm background data in specific areas of interest. The most commonly used include soil sampling, mini-piezometers,

low flow measurements, temperature surveys, ground water chemistry and surveys of surface water elevations.

Soil sampling normally involved the collection of samples using a hand soil probe and auger. Samples are usually taken to a shallow depth (approximately 1.6 m). The samples are analyzed for soil distribution and texture, and permeability is estimated from these. The technique is most useful as a spot check against existing information. The shallow depth employed limits the ability to infer infiltration potential. In some cases the technique is expanded to include the digging of test pits, to greater depth, using a backhoe.

Mini-piezometers are typically 2 m lengths of polyethylene tubing, slotted and screened at one end. They are often installed in areas where the water table is shallow. Measurement of the hydraulic head in the mini-piezometer compared to the adjacent stream allows an assessment to be made as to whether there is a potential for the groundwater to be contributing to the stream's baseflow. Mini-piezometers cannot differentiate between perched and hydraulically connected aquifers, due to the shallow depth of installation.

One of the simplest methods of assessing the importance groundwater in relation to baseflow is to measure stream flow at various locations during prolonged dry periods. The flow measurements, coupled with knowledge of the contributing drainage area, can provide insight into the relative significance of baseflow contribution from different areas. Care must be taken in assessment as farmers ponds, storm water ponds and wetlands may provide a source of baseflow which is not completely attributable to the ground water regime.

Temperature surveys and the identification of spawning redds as part of fisheries investigations can often provide useful information on the distribution of groundwater discharge. Mapping of the temperature by tributary (especially in headwaters areas) can reveal the pattern of groundwater discharge. The pattern of groundwater input can be masked by solar warming in areas without canopy or where there are on-line pools or ponds. Winter surveys, which detect warmer groundwater discharge areas, are sometimes more useful than summer surveys for this reason. Snow cover can limit the use of winter surveys, however.

Ground water chemistry can sometimes provide useful information when correlated with surface water chemistry. In particular, soluble parameters such as nitrates and chlorides may indicate areas of ground water contribution. The difficulty with this technique is that collection of samples from existing wells and other available locations may yield results which are a composite of overburden and deeper aquifer layers. The composite nature of the sample may mask any possible correlations.

The final non-intensive technique which is often utilized is a survey of the water surface of ponds and wetlands in the area. The information collected can be used to estimate water table divides and infer the direction of ground water movement. As with the other techniques

discussed, care must be taken in interpreting the data because it is difficult to differentiate between locally perched water tables, overburden water tables and hydraulically connected systems which receive contribution from deeper aquifers.

Intensive Methods

More intensive geologic and hydrogeologic techniques are available for use in situations where knowledge of the interaction between ground water and surface water is important to the protection of natural systems. Intensive investigations will often be costly and will be complex in design. Individual programs aimed at the conditions of particular areas should be designed by the consulting team's hydrogeologist, in consultation with team biologists and hydrologists. A typical intensive investigation will involve the drilling of nests of boreholes (to determine the overburden and bedrock geology and hydrostratigraphy) installation of piezometers in the different hydrostratigraphic units (to establish water levels and hydraulic gradients), collection of water chemistry data from the different units, and pump tests to establish hydraulic characteristics (transmissivity, storativity, hydraulic conductivity) and the degree of hydraulic connection between aquifer systems. In some cases, other techniques using geophysical methods (electromagnetic surveys, seismic refraction surveys, geophysical borehole logging) have been used to provide supplementary information and to focus in on the areas where drilling is to be completed.

Aquatic Studies

In most watershed and subwatershed studies conducted to date, aquatic studies have focused on fisheries habitat and the fish community, as indicators of the health of the aquatic community. The studies are normally conducted in accordance with guidelines which have been prepared by the Ministry of Natural Resources.

The level of effort applied to aquatic studies and whether they are required or not is determined by an assessment of stream's value, or potential value, as a fisheries resource. This determination is based on the study goals, agency and public input, the background review and preliminary field reconnaissance. If fisheries studies are undertaken, the major difference from one watershed plan to another will be intensity of the study. The techniques employed are similar regardless of the subjective value placed on resource. Two examples of surveys conducted on watersheds of similar size (15 - 25 km^2) are provided below to illustrate the different levels of study deemed appropriate in the specific cases.

Study Characteristic	Stream A	Stream B
General Classification	Bait Fishery	Cold Water-Brook Trout
Habitat Survey	Yes	Yes

Reaches Surveyed	5	34
Sub Reaches Surveyed	2/reach	2/reach
Fish Inventory	Species identification (5 reaches)	Species Identification plus trout biomass (17 reaches)
Benthic Survey	Qualitative	-
Spawning Survey	-	All cold water reaches

The surveys undertaken most often are the habitat survey and the inventory of the fish community. Spawning surveys, benthic surveys and other specialized surveys are conducted according to the needs of the plan.

Habitat Survey

Habitat surveys focus on the mapping and measurement of physical habitat attributes. The attributes include geomorphological features (number of pools, riffles, flats and runs), channel shape, flow characteristics, bank stability (and erosion), substrate characteristics, instream cover and canopy cover. Where stream rehabilitation for fisheries enhancement and assessment techniques such as Habitat Suitability Models (HSI) are contemplated, the detail of the inventory required increases. In addition to mapping, quantification of many of the attributes is required for HSI models.

Fish Community Inventory

Fish inventories are conducted by partitioning off a section of the stream and then capturing the fish present, using one of several techniques, the most common of which is electrofishing. Specimens are captured, identified and counted prior to release in order to establish the species present and their distribution. In studies where rehabilitation or enhancement of the stream is contemplated, specimens of the selected indicator species are measured and weighed in order to establish biomass and life stage distributions.

Terrestrial Studies

Terrestrial surveys and inventories of vegetation and wildlife are conducted primarily due to the protective orientation of the plan (preserve and protect important natural features) and the goals which address the need for corridors and linkages for both wildlife movement and human enjoyment. In watersheds and subwatersheds where there has already been large-scale conversion of natural areas to agricultural use, the inventories are limited and concentrate on the delineation of constraints such as buffers and setbacks and on potential linkage routes, which if preserved or rehabilitated, could connect an isolated natural area (such as a woodlot) to the river valley system. In watersheds or subwatersheds with substantial natural areas such as forests or wetlands, greater efforts are applied in order to collect the information necessary for

classification of the vegetation community so that key areas may be identified and decisions made regarding encroachment or crossings of the natural area (eg. by roads, sewers, etc.).

In general, vegetative inventories conducted under watershed and subwatershed plans are not intended to be exhaustive nor are they intended to seek out rare plants (although these may be found through incidental observation). The same is true of wildlife inventories which may be conducted. The emphasis is on identifying larger significant units and assessing the value they provide in terms of habitat and aesthetic resources.

Vegetative Inventories

The most common form of vegetative study undertaken is involves ground truthing of information derived from aerial photography and previous reports. Major overstory, understory and ground cover species associations are documented. In most cases, this type of study is conducted over a short time frame and therefore information on seasonal plant communities cannot be collected. This level of effort is the most common one associated with watershed plans.

More detailed studies are conducted at the subwatershed plan level, where large significant natural areas have been identified. In such cases more thorough inventories are conducted to characterize individual vegetative units according to physiognomy and species dominance criteria. These studies are normally conducted during the spring, summer and fall in order to identify seasonal differences in the community structure.

Wildlife Inventories

Watershed and subwatershed plans conducted to date have generally limited wildlife inventories to incidental and opportunistic observation during fisheries and vegetative field studies. In some cases transect surveys may be warranted to confirm the presence and range of previously reported rare species.

A.3.3 Models and Other Assessment Techniques

A variety of models and assessment techniques may be required during the development of a watershed/subwatershed plan in order to examine the effects of proposed management options and development on the key physical and biological systems. The selection of the models and assessment techniques to be used will depend on the level of detail of the plan, the role or importance of particular physical systems on resources, and the availability of data to formulate and run the models. Decisions regarding the tools to be used in assessment should be made during the background review stage, because the use of particular techniques may require the collection of specific data during field studies.

Surface Water Models

The types of surface water models commonly employed in watershed and subwatershed plans include a range of public domain hydrologic models and usually, the HEC-2 model for river hydraulics. The actual selection of models must be determined based on the needs of the particular watershed/subwatershed study. In most watershed plans or large subwatershed plans (eg. plans where strategic guidance is not available from higher level plans) these will usually be a need for the combination of models selected to be able to address a number of key areas. These include:

- ability to simulate both low and high flows.
- ability to reflect changes in evapotranspiration and shallow groundwater flow, caused by changes in land use.
- ability to examine flow velocities and durations.
- ability to reflect the effects storage (in water quality or quantity facilities) on the duration of flows.
- ability to examine seasonal flooding (needed for biological reasons) in addition to low frequency flooding.

The need for the combination of models to be able to deal with these areas is dictated by the types of impact which may be expected from urbanization or other land use change. The primary surface water issues are usually baseflow or low flow (for fisheries), stream erosion (fisheries and damage), and periodic flooding (nourishment of wetland habitats).

In order to meet these assessment requirements it is expected that in most cases a hydrologic model capable of continuous or semi-continuous simulation will be required. Event models, of the type used traditionally for flood and stormwater quantity assessments, are generally insufficient because they do not permit evaluation of the flow regime over the range of low and medium flows which are important to the fisheries resource and erosion concerns.

The model(s) selected needs to be amenable to use in water quality assessments but generally do not need to include a specific water quality simulation capability. Most water quality modelling is carried out using relatively simple techniques involving mass balance or utilizing specialized models to examine such parameters as temperature and dissolved oxygen. In general, this level of water quality analysis is sufficient for management decisions and is all that is warranted by the available data base.

Groundwater Models

Groundwater modelling, as it is conducted in the majority of watershed and subwatershed plans, usually takes the form of a regional water balance which estimates the expected infiltration based on total precipitation, losses to evapotranspiration, runoff and increases in the soil moisture

storage. The method has the advantage of requiring relatively little data, but is limited by the difficulty in estimating the soil moisture storage potential and the inability of the technique to characterize complex systems where there is interaction between shallow and deep aquifers. Despite these difficulties, water balance type models are often the most practical, at the scale of a large watershed or subwatershed, because the cost of collecting the data necessary to formulate and calibrate more sophisticated models is prohibitive.

At the detailed subwatershed plan level, more sophisticated groundwater models such as MODFLOW (USGS, 1988) may be warranted. Careful consideration of the need for this form of modelling must be undertaken because of the relatively high costs associated with the collection of the data (hydraulic parameters, stratigraphy) needed to set up and run the model. In some cases, especially where the effects of urbanization must be characterized in order to protect valued wetlands or water supplies, it will be important to examine the impacts of development on the depth of the water table and there will be no alternative. In such cases a detailed and rigorous understanding of the geology and hydrogeology is needed and models, such as MODFLOW, will be required to determine the long term or cumulative impacts of development.

Habitat Suitability Models

In many watershed/subwatershed studies there is a need for the effective communication of concepts like fish habitat quality and quantity between biologists and other disciplinary specialists. Furthermore, a tool for assessing environmental quality and integrating the effects of environmental change on fish habitat quality is required. The Habitat Suitability Index (HSI) model is such a tool. It produces easily understood habitat index values which can be readily communicated to other disciplines and which can be used to compare habitat quality between locations. Through linkages with other models, the HSI can be used to predict the effects of environmental change on fish habitat quality resulting from alternative land use scenarios and/or habitat restoration programs.

Habitat suitability index models were developed by the U.S. Fish and Wildlife Service to: (1) provide a consistent method of processing and summarizing habitat data for use in watershed management decisions; and (2) promote standardized data collection. Many HSI models have been developed for individual fish species (e.g., Brook Trout -- Raleigh 1982). Species specific HSI models use habitat variables that are important for each life stage. The models usually produce a single overall measure of habitat suitability for the species, but can be modified to produce indices for each life stage. Effects of environmental changes on fish habitat quality may be explored by changing input parameters in the model.

HSI modelling is a habitat-based assessment technique. It is sometimes preferred over community-based techniques which, because they are based on biological systems, are highly variable, must be measured several times over a season, and can be complicated by interactions

such as density dependence and predation. The major advantage of HSI techniques is that, if properly applied, they provide a defensible basis for predicting the effect of changes to various habitat attributes.

In HSI modelling, suitability index graphs, specific for each habitat variable, are used to transform raw habitat data into suitability index values between 0 and 1, with 0 being poor habitat and 1 being optimal habitat. Through a series of arithmetical (e.g. sum, product, mean) or conditional (e.g. extreme) aggregation and transformation steps involving two or more variables, a single overall habitat suitability index value is derived.

In watershed scale applications, single composite HSI values are generated for each area of interest, for example, a tributary. Essentially, this composite value is an index of overall habitat quality; the higher the value the better the habitat quality. This is an simplistic approach which can mask some important information, but it is generally sufficient for policy level management decisions.

At the subwatershed level of planning it may be necessary to formulate the HSI models at a finer level of detail. For example, composite indices will not explain why one area might have poorer habitat quality than an adjacent area, or alternatively, what habitat type is limiting production. What is required is an index of habitat quality for each life stage (e.g., spawning habitat, nursery habitat, etc.). Such information is essential if one wishes: (1) to determine the most effective approaches to habitat restoration; (2) to properly evaluate the consequences of various development scenarios; (3) to determine which areas require the greatest level of protection; or (4) to determine where habitat potential is greatest. To illustrate the above, it is possible that two areas with different habitat attributes could have identical composite HSI values, and yet in one area spawning habitat might be limiting production, whereas in the other area, nursery habitat may be limiting. Attempts to increase the productive capacity of fish habitat in the area which is limited by nursery habitat by providing additional spawning habitat would be ineffective. For this reason, life stage specific HSI index values are required. These and all other HSI indices must be interpreted carefully and with recognition of the fact that fish move (i.e., the lack of spawning habitat in one area may not be a problem if suitable spawning habitat is available nearby).

Possible limitations of HSI models include: (1) mis-specification of the model (variables may be unimportant, or confounded with others, or not included); (2) the inability to produce index values correlated with fish standing crop or biomass; (3) models have been validated for only a few watersheds; (4) many models are based upon subjective, albeit expert, opinion; and (5) models were primarily developed for inter-watershed comparisons. These limitations must be considered carefully in applying HSI modelling to specific watersheds/subwatersheds.

Terrestrial Resource Assessment Tools

Part of the watershed/subwatershed planning process is the determination of which natural areas are the most valuable or sensitive and require protection from developmental pressures. These lands may include areas that are valuable or required wildlife habitat, areas that are representative of a vanishing vegetation type, areas which provide habitat for rare species, or areas that provide a vital link between natural areas.

At the watershed plan level the emphasis will usually be on the protection and linking of major blocks and key resources, such as ESAs, ANSIs, major wetlands and woodlots, and the overall stream valley/corridor. At this level it is possible to make strategic management decisions using a relatively informal assessment, relying primarily on professional judgement and previous formal designations.

At a subwatershed level, where less well documented terrestrial resources will receive a greater focus, a more formal evaluation procedure may be used to identify and prioritize natural areas for zoning protection. The need for such a procedure will depend largely on the direction given by the watershed plan, the nature of the subwatershed, and the management decisions which may be contemplated. A formal procedure may be particularly important if areas are being considered for protection/preservation which are on the tableland and not in flood or hazard areas (eg. in private ownership). In such cases, strong justification for the protection of these areas will be needed.

Such a formal evaluation method must consider both the context and content of each natural area (Noss 1987). Context factors such as the surrounding landscape and proximity to other natural areas need to be included in an evaluation of an area, as well as content factors such as floral and faunal populations, diversity, and habitat quality. One such system which is used in Alachua County, Florida, and based on criteria incorporating both site content and site context factors, is easily weighted for different planning objectives (Deuver and Noss 1989). This particular system has been adapted for use in the Hanlon Creek Watershed (MMM and LGL, 1991). Other similar systems could be used, but this particular one requires that the different areas identified be ranked using six criteria:

1) **Vulnerability**, based on our knowledge of human community development patterns and local development pressures.

2) **Rarity**, incorporating the rarity of each site's plant and animal community types, the rarity of the species for which the site provides habitat, and the uniqueness of the site's special features. Rarity is evaluated on four levels: county, regional, provincial and national.

3) **Connectedness**, which is an indicator of how the site links, or could potentially be linked with related elements of the landscape, such as other natural upland areas, significant wetlands or ESAs. The site's habitats are considered relative to other nearby natural areas, and the possible functions of a site, such as a natural linking corridor or buffer for another natural area will be considered.

4) **Completeness**, is an index of a natural area's ecological quality. This criterion combines information on habitat and species diversity with site condition in terms of environmental degradation. This criterion also contains a measure of the size of the natural area under consideration.

5) **Manageability**, involves an assessment based on long-term viability. The score provides an index of the restoration potential, size and compatibility with neighbouring land uses for each natural area.

6) **Nature-oriented human use potential**. This criterion is included because of the urban nature of the lands included in most watershed/subwatershed plans, and high use pressures likely to develop on natural areas surrounded by residential developments. This criterion indicates a natural area's inherent suitability for human non-destructive use of its natural features.

For each of these criteria, values are assigned on a scale of 1 to 5. In order to give an accurate overall assessment of each natural area, the scores are weighted to give extra emphasis to the range of species and habitat types represented (completeness), and to the size of the area being evaluated. Because size of an area is so important for many species, size is used to give extra weight to large areas with a great diversity of habitats and species. This weighting helps to balance the scores so that the highest ranking areas will not only support rare species, but will also be excellent representative areas.

Site-specific information is used for the evaluation of terrestrial resources, based on background information, airphoto interpretation, and data collected in field studies. Natural areas within the subwatershed are identified and described using floral species association lists and wildlife use information.

APPENDIX B

SUBDIVISION / SITE PLANNING EXAMPLE

An Example of Subdivision / Site Planning

Proposed Residential Community in the Town of Aurora

A 200-unit residential development proposal is located in the Town of Aurora within the Oak Ridges Moraine Area. The 45 hectare site is situated at the southwest corner of Bayview Avenue and Vandorf Sideroad. Part of the site falls within an Environmentally Significant Area as designated by the Lake Simcoe Region Conservation Authority. The project is proposed to be serviced with full municipal water supply and sewage systems.

This example will demonstrate how environmentally responsible planning ideas are applied to the site. The fundamental, and most important principle in this approach is the recognition at the outset that the environment will dictate the location of development. The desired population density for this area will then dictate the form of the development.

The Planning Process - A Decision Making Procedure based on Environmental Sensitivity

A full understanding of the ecosystem and the environmental issues is obtained through the preparation of a series of technical studies. These include the documentation of all the natural resources information collected from available sources and directly from the field, a hydrogeological study, a surface water management study, fisheries/woodland/natural area management studies as well as a municipal servicing report (Marshall Macklin Monaghan, 1993). The planning process which leads to the formulation of the development concept can be summarized in three simple steps:

- the concept evolves from a full understanding of the ecosystem and the interrelationship amongst the different environment components
- the establishment of an environmental enhancement strategy for the site
- the determination of a sensible development approach, built forms and densities

To restate this planning process simply: the environment dictates the location and layout of development.

The Site

The 45 hectare site includes a continuous wooded valley along the central part of the property extending from north to south. It is strategically located as a major linkage between the White Rose West Forest Life Sciences Area of Natural and Scientific Interests (an ANSI defined by the Ministry of Natural Resources) south of the property and the wooded valleyland to the north.

This densely wooded valley bisects the site and a branch of the Holland River headwater flows from south to north along the valley. The areas on both sides of the woodland are abandoned farm land with rolling relief and varied topography. This undulating landscape on the western side of the valley has created two distinct ridgelines running in a north-south direction. Part of the site is also designated by the Lake Simcoe Region Conservation Authority as an Environmentally Significant Area (ESA) due to the function of the site as an aquifer recharge zone and stream recharge zone.

Farming as an intensive activity in the past had disturbed the natural environment significantly. Many of the existing features within the site reflect the impact of previous agricultural activities upon the natural environment. Some of these features include:

- straight and geometrically shaped woodland edges with restricted bio-diversity
- clearance of the original woodland and vegetation along streams and watercourses
- disturbed fish habitat
- fragmented clusters of regenerated vegetation with no connecting corridors
- exposed watercourses and warming up of the surface water
- high level of nitrate concentration in the ground
- old farm access lane cutting across the wooded valleyland and obstructing fish passage

An Environmental Enhancement Strategy for the Site: Assist Nature to Reclaim

An environmental enhancement strategy is developed for the site. The strategy is to improve and enhance the environmental quality and diversity of the ecosystem through the adoption of a sensible and responsible development form within the site, combined with corrective measures to restore areas and features of environmental degradation. The following basic planning principles are developed and they represent a departure from the conventional residential subdivision planning approach:

- Recognizing that past activities and abandonment have created situations of natural degradation within the site, development projects should be taken as opportunities to enhance the environmental qualities of the land by incorporating positive measures to encourage natural regeneration and habitat restoration, promote habitat connectivity and assist nature to reclaim;

- Cluster housing forms, non-freehold tenure and responsible site planning criteria should be adopted to achieve a high level of environmental efficiency within the site. A development density similar to conventional freehold subdivisions is attainable while existing landform and natural areas are conserved;

- An environmentally friendly storm water management approach which encourages infiltration and enhancement of runoff quality should be adopted. Innovative storm water management practices should be incorporated to replace conventional runoff collection and concentration facilities;

- The development of a new residential community should include design elements which promote environmental education and interpretation for the residents. Neighbourhoods should be designed to maximize the opportunities to appreciate the inter-relationship between human and the natural environment (ie. man is a part of nature, not separate from it).

Sustainable Development: Site Specific Planning and Design Criteria

Based on the environmental enhancement strategy, the proposed residential development has adopted 19 site specific planning criteria. A development plan for the residential community is prepared. These criteria are:

Protect ecological integrity

- *protect existing woodland as part of a greenway system.*
- *respect existing natural topography and features*

Create new habitats and encourage natural regeneration

- *assist natural woodland edge regeneration*
- *create woodland corridors and enhance habitat connectivity*
- *enhance riparian vegetation along watercourses*
- *enhance existing wetland*
- *protect fish passage*
- *re-vegetate environmentally degraded areas*
- *encourage natural rejuvenation of disturbed watercourses*

Conserve landform and natural significant areas

- *identify major ridgelines and slopes for protection*
- *adopt cluster housing form, condominium ownership and creative site planning standards*
- *retain existing rural character*

Preserve and enhance groundwater resources

- *adopt site specific storm water drainage approaches to reflect local soil conditions*
- *promote infiltration and replace curbs and gutters with roadside infiltration ditches and check dams*
- *incorporate infiltration trenches within rear yards*
- *respect natural topography in overflow design*

Promote environmental education and interpretation

- *provide direct trail connection to the town-wide trail network*
- *develop the local park as a nature park based on ecological principles*
- *use native plant species in planting along local streets and within amenity areas*

The Three Tests for Sustainable Suburban Residential Development: Municipal Land Use Policies, Development Control / Design Performance Standards, and the Market

The proposed residential community in the Town of Aurora has received very supportive response from the Ministry of Natural Resources and the Lake Simcoe Region Conservation Authority. They see the proposal compatible with the fundamental principles outlined within the Implementation Guidelines - Provincial Interest on the Oak Ridges Moraine Area of the Greater Toronto Area. The proposal has established site specific planning and design criteria in accordance with the principles of sustainable development. In contrast to the widespread misconception that no development will ever be "approvable" within the moraine area under the Implementation Guidelines, the development plan has demonstrated that there is an alternative to the conventional suburban single detached freehold subdivision. Nevertheless, there are three further tests beyond the endorsement from the Ministry: municipal land use policies, development control / design performance standards, and the market.

- **Municipal land use policies**

  Cluster housing forms (townhouses, courts, walk-up apartments) and non-freehold (condominium or leasehold) ownership are commonly adopted by the development industry for projects within urban areas. Nevertheless, these are not traditional land uses commonly found in suburban fringe areas within the GTA. The project has illustrated that an environmentally efficient development form can be achieved without sacrificing development densities based on these non-conventional land uses and built forms. Interpreting this approach on a broader regional planning perspective suggests that there is a way to achieve a balance between the public objective to promote sustainable development and the accommodation of settlement growth in urban fringes.

  However, cluster housing form and condominium ownership on a significant scale are

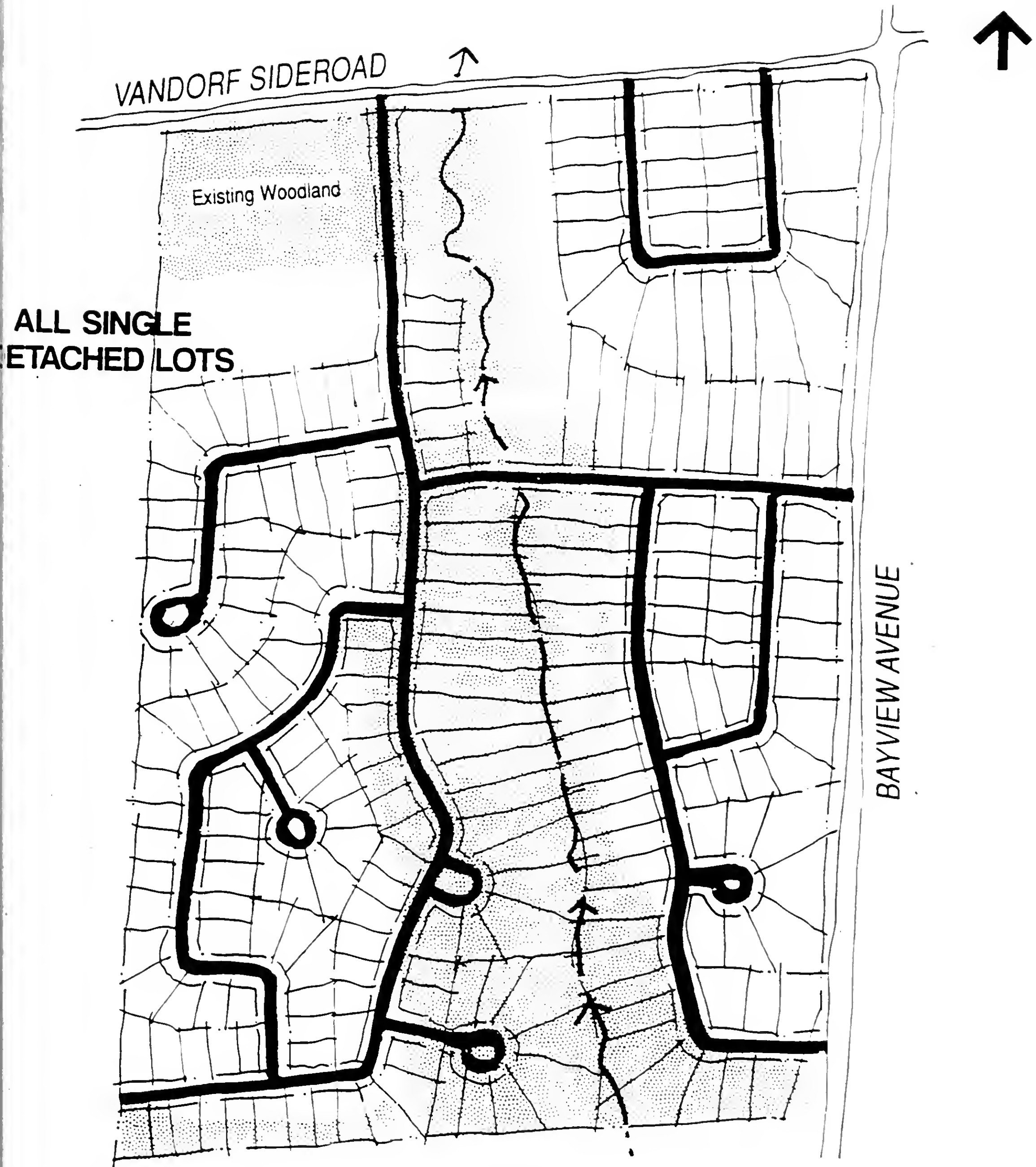


Figure B.1 Typical Subdivision Layout

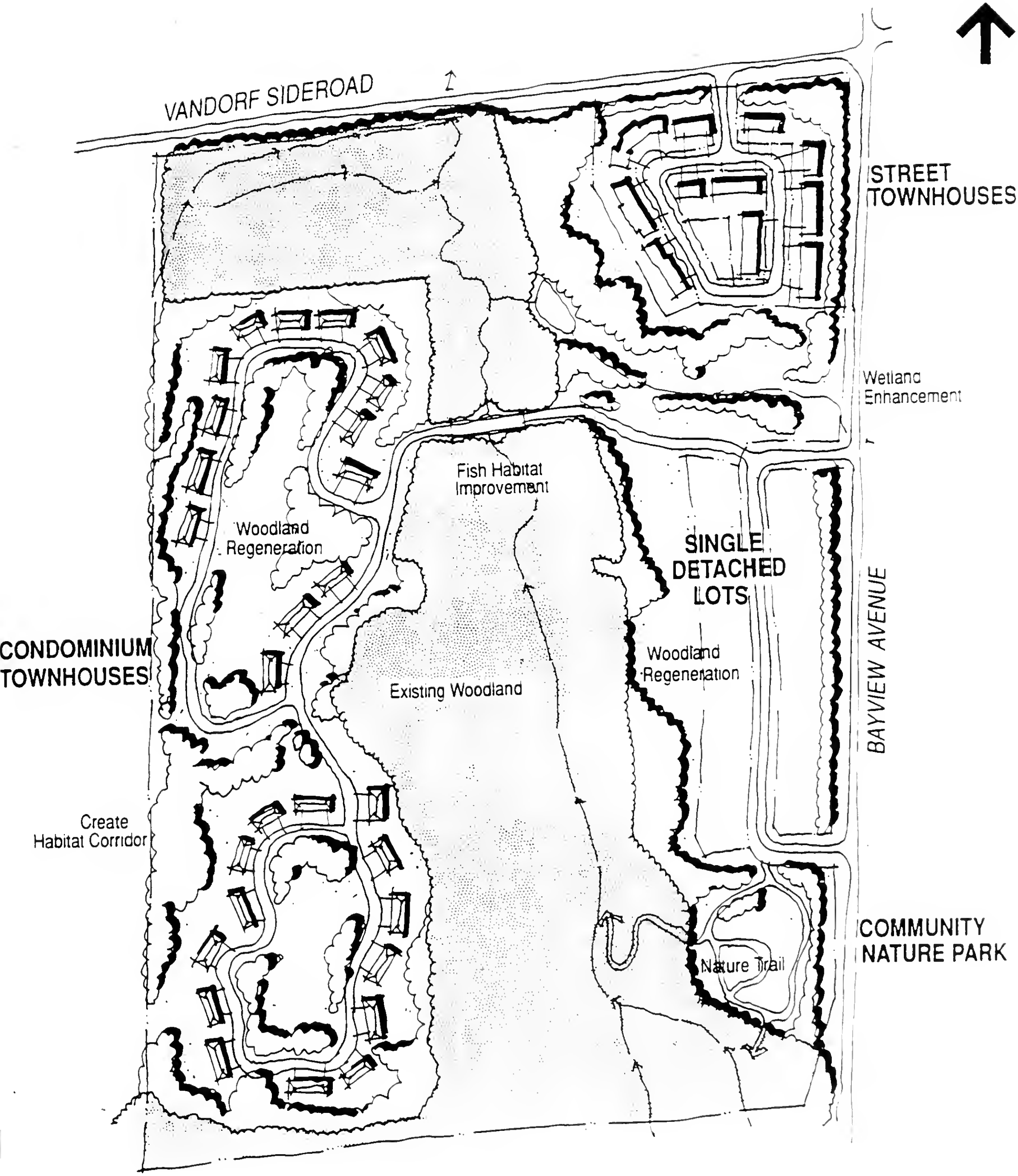


Figure B.2 Subdivision Layout Implementing Subdivision/Site Planning

normally the least understood by local residents, municipal politicians and municipal staff. Most local municipal official plans have included specific policies for estate residential development and suburban freehold residential development. These policies stipulate, in detail, the densities, scale, minimum lot sizes and frontages for the development. They seldom include specific land use planning policies to encourage the use of compact built forms and ownership as the means to promote sustainable development. Municipalities should take a pro-active role by including enabling policies within their official plans.

- **The Market**

Developers are familiar with the response that the typical suburban residential products in the Greater Toronto Area is a two storey single detached home with a double garage, sited within a freehold lot. Is there a market for the innovative condominium cluster housing located within urban fringe areas? Recent market research undertaken for different various similar proposals indicate a strong current demand for reasonable priced dwellings which are maintenance free, close to existing small towns, themed, and sited within an attractive environment. They appeal to specific sectors of the housing market such as the empty nesters and the young families as potential buyers within the Greater Toronto Area. While some of the projects which are currently planned to capture this specific market will turn out to be successful, the depth of this market is yet untested.

- **Development Control and Design Standards**

The proposed residential development has included site specific site planning criteria, innovative storm water management practices, private access road standards, flexible site grading approach and non-conventional municipal road cross sections. Discussions are being undertaken to request the Town to accept the proposed modifications to the existing municipal design standards (such as the replacement of the curbs and gutters by roadside ditches).

It is important to note that the project adopts non-freehold tenure because this is currently the only way to achieve a higher level of flexibility in the planning and design of the site. Non-freehold ownership arrangement (such as leasehold and condominium development) allows the project to free itself from established municipal public right-of-way and grading design requirements. This flexibility in turn permits the project to achieve its environmental objectives. It has been demonstrated that the adoption of flexible engineering design approach and creative site planning criteria is necessary to accommodate settlement growth while protecting the natural environment. The draft Regional Municipality of York Official Plan (September 1993) has explicitly included site design policies to encourage innovative site planning practices.

However, adopting alternative land ownership approaches as the only means to achieve sustainable development objectives is restrictive. It is timely that local municipalities should incorporate flexible site engineering, grading and road design criteria for areas of environmental significance and permits the application of these flexible standards irrespective of the type of land ownership.

Save What Needs to be Saved, and Build What Needs to be Built

The proposed residential community in the Town of Aurora represents a case study which illustrates how a specific development proposal practises the principles of sustainable development. The project is a major step towards sustainable development within the Oak Ridges Moraine Area of the Greater Toronto. The development process, the form and its planning criteria differ significantly from the conventional subdivisions. The success of this planning approach on a broader land use planning perspective is yet to face three tests: municipal land use policies, development control/design performance standards, and the market. It nevertheless demonstrates that " confrontational situations between town officials, developers and conservationists can be averted by a balanced approach which ... saves what needs to be saved, and builds what needs to be built " (Massachusetts Department of Environmental Management, 1990).

APPENDIX C

PRECIPITATION ANALYSIS

Use of the 4 hour Chicago Distribution 25 mm storm

The 4 h Chicago storm is referenced throughout the document for erosion control and the sizing of first flush pipes, forebays, etc. The 4 hour Chicago storm was used since it is one of the most widely accepted storm distributions across the province which is applicable for urban areas.

A 4 hour distribution was used instead of a 2 hour distribution since the 2 hour distribution produces unreasonably high peak flows (ie. similar to a 2 year storm). Given that stormwater quality measures are intended for the everyday event, the 2 hour distribution is excessive for design purposes. If for example, if the minor system is designed for a 2 year flow, a first flush pipe which accepted the 2 year peak flow would not be appropriate.

Figures C.1 and C.2 indicate that a 25 mm event occurs approximately 4 times per year and that a daily capture of 25 mm would result in an annual capture rate of 95% of the annual precipitation.

The timestep for simulation should be based on the representative time of concentration of the site being modelled. The peak flow from a site may not be modelled accurately if the simulation timestep is larger than the time of concentration since the peak flow could occur within a timestep and may be missed.

Previous work (Urban Drainage Design Guidelines, 1987) indicates that the discretization of a Chicago distributed hyetograph with a timestep less than 10 minutes results in unrealistically high peak flows. In situations where a timestep less than 10 minutes is required, the 10 minute intensities can be replicated based on the timestep required.

Use of a 20 mm capture

The 20 mm storm capture rate was used for rooftop leaders since this is equivalent to a 90% annual precipitation capture rate as shown on Figure C.2. Capturing more runoff than 20 mm for infiltration type SWMPs results in an excess expenditure of money for storage which, for the most part, will not be utilized.

Use of the 15 mm storm

The use of a 4 hour Chicago distribution of a 15 mm storm is proposed for the temperature impact assessment for wet ponds and wetlands in Chapter 4. An analysis of 20 years of the daily precipitation record from the Atmospheric Environment Service rainfall gauge at Yonge and Bloor Street indicated that there are approximately 14 days a year which have a daily precipitation depth greater than 15 mm (Figure C.1), whereas there are approximately 130 days a year which have a daily precipitation depth less than 15 mm (excluding days with no precipitation). Therefore the use of the 15 mm storm is more realistic in assessing a change to

the overall thermal regime in the receiving waters.

This storm was also chosen as a reasonable compromise for SWMPs which would not be economically feasible, or would be adversely affected, if designed to accommodate the runoff from a 25 mm storm. These SWMPs include sand filters, infiltration trenches and basins, and pervious pipes and catch-basins.

Use of the 10 mm storm

An analysis of 20 years of the daily precipitation record from the Atmospheric Environment Service rainfall gauge at Yonge and Bloor Street indicated that a daily capture of 10 mm is equivalent to a annual capture rate of 70% (70% of the precipitation is treated).

This storm was chosen as a reasonable compromise for SWMPs which would not be economically feasible, or would be adversely affected, if designed to accommodate the runoff from a 25 mm storm.

These SWMPs include vegetated filter strips and oil / grit separators.

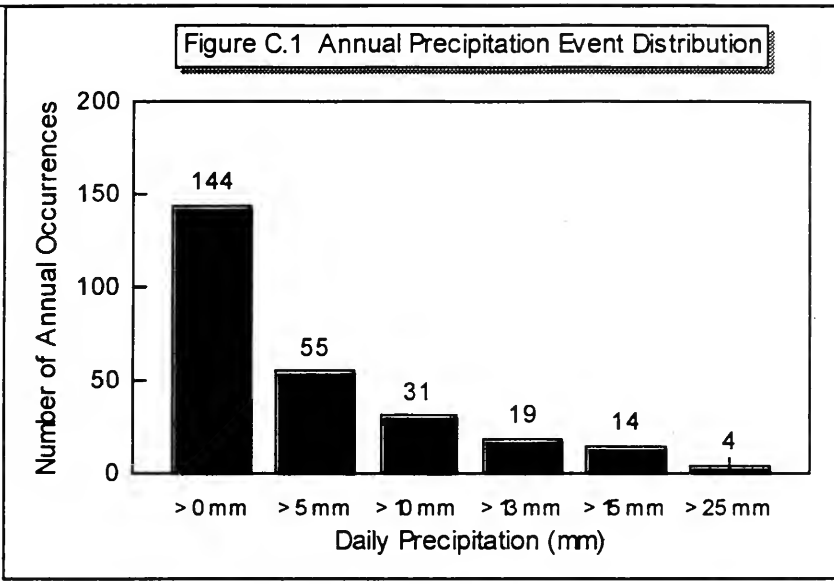


Figure C.1 Annual Precipitation Event Distribution
Number of Annual Occurrences
200
150
100
50
0
144
55
31
19
14
4
> 0 mm
> 5 mm
> 10 mm
> 13 mm
> 15 mm
> 25 mm
Daily Precipitation (mm)

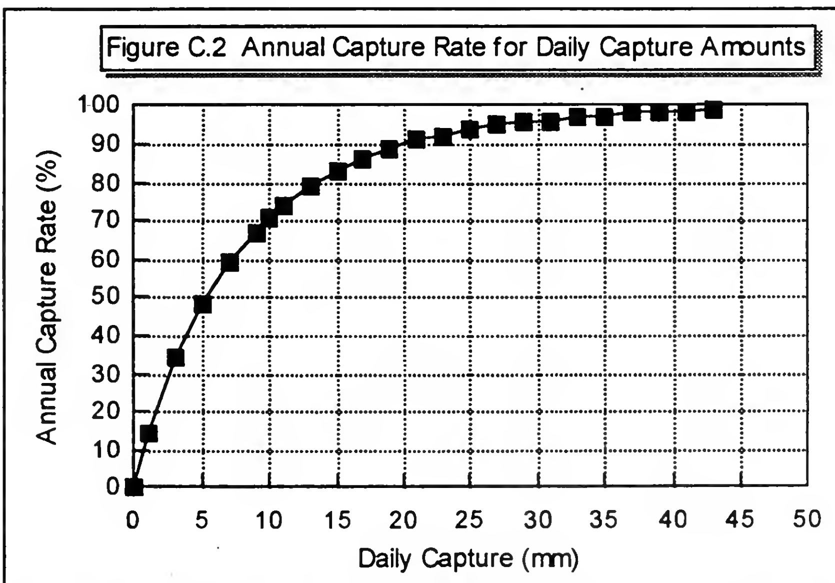


Figure C.2 Annual Capture Rate for Daily Capture Amounts
Annual Capture Rate (%)
100
90
80
70
60
50
40
30
20
10
0
0
5
10
15
20
25
30
35
40
45
50
Daily Capture (mm)

APPENDIX D

FROST HEAVE POTENTIAL

Frost Heave

Figure 3.5 was based on professional judgement regarding the potential for frost heave as a result of the implementation of subsurface infiltration trenches/pits.

Water expands by 9% when it freezes. In a confined trench the increase in volume will result in an increase of trench depth by 9% (ie. the trench will heave upwards equal to 9% of the water depth in the trench at the time of freezing).

Depending on the type of surrounding soils and storage media in the trench itself there is the potential for the formation of ice lenses which can significantly increase the depth of heaving in addition to the expansion of the water in the trench during freezing.

Ice lenses form by the capillary action of water from the water table rising into the pore spaces of soil in the frost zone. The water which migrates to the pores freezes adding to the volume/depth of heaving. Depending on the soil conditions and location of the trench with respect to the capillary zone this migration/freezing could potentially cause significant heaving.

However, ice lens formation is not anticipated to be a problem given the recommended storage media to be used in the trench, and the native soil conditions under which it is recommended to implement subsurface trenches.

Ice lens formation is prevalent for silty soils. These soils have small pores in which water is held but does not freeze, allowing the migration of water through the capillary zone to the frost zone. Frost heave is not prevalent in sandy soils since the pore space is too large and water contained in these pores will either drain quickly or freeze preventing the migration of capillary water. The small pore spaces in clays usually restricts the actual rate of movement of water also limiting the potential for the formation of ice lenses.

Subsurface trenches are generally recommended for sandy soils, and under these conditions frost heave will not be a major problem. Clear stone is recommended as the storage media in trenches. The pore spaces in stone will freeze, preventing the migration of capillary water. In addition, the pore spaces in the clear stone are large enough such that when water freezes in the trench the stone will rearrange itself limiting the expansion heave.

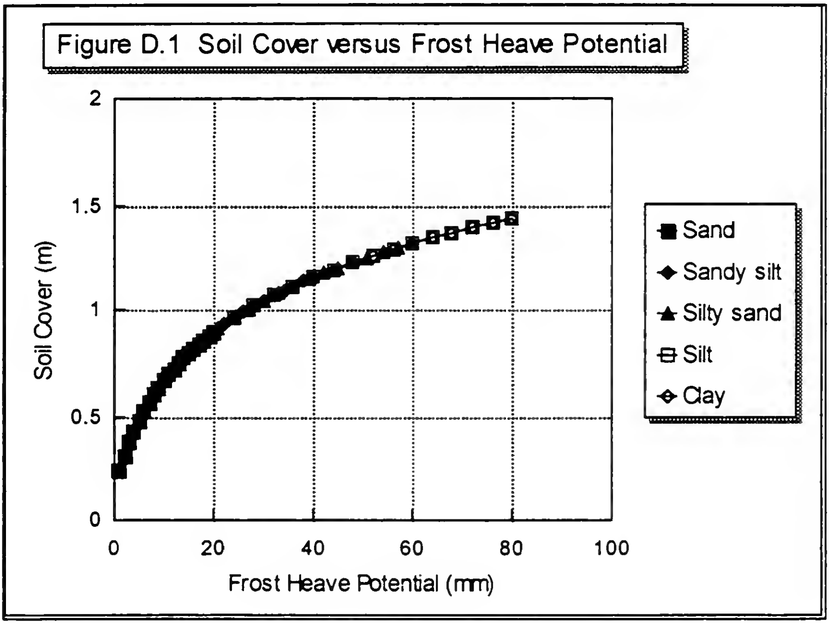
Consequently, frost heave is not anticipated to be a major problem with soakaway pits and trenches unless the soils are questionable for infiltration. In these instances, there will be frost heave in the surrounding soils as well as the trench. Recognizing this, the surrounding native soil material and depth of water in the trench were used to derive a relationship for soil cover based on these factors.

The resulting relationship (Figure 3.5) indicates that the soil cover for silty sands and silts approaches frost cover quickly as the depth of trench (and hence, water in the trench) increases. The soil cover for sands does not increase as rapidly given the lower potential for water to be

available for freezing and the lower potential for capillary rise of water.

Assumptions were made concerning the volume of water availabe for freezing in the trench for the various native soil conditions :

Sand	25 % of trench filled with water
Sandy silt	50 % of trench filled with water
Silty Sands	75 % of trench filled with water
Silt	100 % of trench filled with water
Clay	100 % of trench filled with water



Silt and clay soils were included for completeness, although, it is not anticipated that infiltrationwill be promoted in the majority of cases where these soil conditions occur.

These assumptions were used to derive a relationship between the soil cover and potential frost heave depths. This relationship is shown in Figure D.1 and is the basis for Figure 3.5. It was assumed that the heaving translated to an increase in depth only (as mentioned before). As the frost heave approaches 25 mm, the soil cover rapidly increases to minimize heaving. In situations where there is the potential to have heaving greater than 25 mm, the soil cover

approximates the frost depth (1.2 to 1.5 m below the ground surface) negating or minimizing any frost heave potential.

Figure D.1 indicates that the relationship between soil cover and frost heave potential is the same for any soil type (as would be expected). The curves in Figure 3.5 are different for the varying soil types because of the different volumes of water which were assumed available for freezing.

APPENDIX E

FALLING HEAD DRAWDOWN EQUATION

Falling Head Drawdown Equation

Equation 3.2 was presented as appropriate for the calculation of drawdown time in extended detention facilities. Equation 3.2 is based on a falling head, and several calculations should be made since the total drawdown time is a function of pond surface area with depth.

In its simplistic form (constant pond area) the equation reduces to an SCS hydrograph approach whereby the average flowrate out of the pond is equal to half of the peak flowrate, and the total drawdown time is equal to twice the extended detention volume divided by the peak flow rate.

The following derivations prove these simplistic rules.

The falling head equation can be written as :

$$t = \frac{2 \quad A_p}{CA_o(2g)^{0.5}} \qquad (h_1^{0.5} - h_2^{0.5})$$

This equation is a simplified version of an integral based on an approximation of constant pond surface area (A_p) through discrete changes in head. If the assumption of constant pond surface area is made, one calculation could be made for the total drawdown time and h_2 would become zero and h_1 is just the total head (h) over the orifice when the pond is full (ie. at the peak outflow from the pond).

$$t = \frac{2 \quad A_p}{CA_o(2g)^{0.5}} \qquad (h^{0.5})$$

Rearranging :

$$CA_o(2g)^{0.5}\ t = 2A_p(h^{0.5})$$

Multiplying both sides by $(h)^{0.5}$

$$CA_o(2gh)^{0.5}\ t = 2A_p h$$

The left hand side of the above equation is the familiar orifice equation multiplied by the drawdown time. At the head h, this is the peak flow out of the pond (Q_p) multiplied by the drawdown time. With a constant surface area (A_p) the right hand side of the equation is simply the volume of extended detention storage (V).

$$Q_p\ t = 2\ V$$

Therefore :

$$Q_p = 2V/t \quad \text{or} \quad t = 2V/Q_p$$

An SCS hydrograph is based on 1/3 of the hydrograph being in the rising limb and 2/3 of the hydrograph in the recession limb. Using simple trignometry the volume of water in the rising limb and recession limb can be approximated by :

$V_{rising} = 0.5\ (1/3t\ \ Q_p)$

$V_{recession} = 0.5\ (2/3t\ Q_p)$

The combined rising and recession volumes equal the extended detention volume in the pond

$V = V_{rising} + V_{recession}$

An average flowrate out of the pond equals

$Q_{avg} = V/t$

Substituting the equations for the rising limb and recession limb volumes into the overall volume calculation produces :

$V = 0.5\ (1/3t\ Q_p) + 0.5(2/3t\ Q_p)$

$V = Q_p t/2$

Rearranging

$Q_p = 2V/t$ or $t = 2V/Q_p$ and $Q_p = 2Q_{avg}$

This indicates that the SCS hydrograph is acceptable if there is a constant surface area in the pond. **In most cases, however, given the grading which is recommended in wet facilities, the assumption of constant pond area may not be appropriate and a relationship should be derived between pond surface area and depth to solve the falling head equation. These calculations are as follows :**

$$dt = \frac{A_p\ dh}{A_o C(2gh)^{0.5}}$$

Using a linear relationship to describe the changes in A_p with y

$A_p = C_2\ h + C_3$

and substituting

$$dt = \frac{(C_2 h + C_3)dh}{A_o C(2gh)^{0.5}}$$

$$\int dt = \frac{1}{A_o C(2g)^{0.5}} \times \int (C_2 h^{0.5} + C_3 h^{-0.5})dh$$

Integrating and evaluating over the total pond depth h (where the starting depth is 0)

$$t = \frac{C_2 h^{1.5} + C_3 h^{0.5}}{A_o C(2g)^{0.5}}$$

if C is set at 0.62

$$t = \frac{C_2 /1.5\ h^{1.5} + C_3 /0.5\ h^{0.5}}{2.75 A_o}$$

or rearranging

$$A_o = \frac{0.66 C_2 h^{1.5} + 2 C_3 h^{0.5}}{2.75\ t}$$

For a 24 hour drawdown time

$$A_o = \frac{0.66 C_2 h^{1.5} + 2 C_3 h^{0.5}}{237600}$$

APPENDIX F

PLANTING STRATEGY - SPECIES LIST

Planting Strategy - Species Lists

Deep Water Areas (1 m ≤ depth ≤ 3 m)

Pondweeds

Sago Pondweed (Potamogeton pectinatus)
Floating Pondweed (Potamogeton natans)
Large-leaved Pondweed (Potamogeton amplifolius)

Others

Water Stargrass (Heteranthera dubia)
Canada Waterweed (Elodea canadensis)
Coontail (Ceratophyllum demersum)
Tapegrass (Vallisneria americana)

Shallow Water Areas (≤0.5 m)

Submerged Species

Pondweeds
Floating Pondweed (Potamogeton natans)
Sago Pondweed (Potamogeton pectinatus)

Others
Canada Waterweed (Elodea canadensis)
Coontail (Ceratophyllum demersum)
Fragrant Waterlily (Nymphaea odorata)
Water Stargrass (Heteranthera dubia)

Emergent Species

American Bulrush (Scirpus pungens)
Common Arrowhead (Sagittaria latifolia)
Pickerelweed (Pontederia cordata)
Reed Grass (Phragmites communis)
Softstem Bulrush (Scirpus validus)

Sedges

- Carex lacustris
- Carex pseudocyperus
- Carex retrorsa
- Carex utriculata

Shoreline Fringe (Extended Detention Areas)

Shrub Species

- Bebb's Willow (Salix bebbiana)
- Meadowsweet (Spiraea alba)
- Pussy Willow (Salix discolor)
- Red-osier Dogwood (Cornus stolonifera)
- Sweet Gale (Myrica gale)

Hydric Grasses

- Bluejoint (Calamagrostis canadensis)
- Cut Grass (Leersia oryzoides)
- Red Fescue (Festuca rubra)
- Reed Canary Grass (Phalaris arundinacea)

Flood Fringe Areas

Tree Species

- Balsam Poplar (Populus tremuloides)
- Black Willow (Salix nigra)
- Eastern White Cedar (Thuja occidentalis)
- Red Ash (Fraxinus pennsylvanica)
- Silver Maple (Acer saccharinum)

Shrub Species

- Common Elder (Sambucus canadensis)
- Red-berried Elder (Sambucus pubens)
- Red-osier Dogwood (Cornus stolonifera)

Round-leaved Dogwood (Cornus rugosa)
Wild Black Currant (Ribes americanum)

Upland Areas

Tree Species

White pine
Red pine
White spruce
Eastern white cedar
Sugar maple
Red maple
Silver maple
Ash sp.
Red oak
Bur oak
Swamp white oak
Common alder
Ironwood
Black cherry
Choke cherry
Black willow
Aspens

Shrub Species

Common juniper
Viburnum sp.
Grey dogwood
Red Osier
Meadow Sweet
Amelanchier sp.
Sumac sp.
Willow sp.

Herbaceous Species

Red fescue

Reed canary grass
Wild strawberry
Jewel weed
Clovers

Pond Berming Areas

Fragrant Waterlily
American Bulrush
Softstem Bulrush

Filter Strips

Red Fescue
Tall Fescue (Festuca arundinacea)
Redtop (Agrostis alba)

APPENDIX G

FOREBAY SIZING - DERIVATIONS AND RATIONALE

Forebay Sizing

The sizing of the forebay is one of the most complicated tasks in designing an end-of-pipe stormwater management facility. The forebay is implemented with two objectives in mind :

- restrict the area of sediment removal
- minimize resuspension of settled material

These two objectives are conflicting. If the area of sediment removal is confined to the forebay, it can be expected that the greatest potential for resuspension is in the forebay. Recognizing this, forebays are typically sized using two methods :

- sedimentation basin theory
- scouring theory

Neither one of these theories is ideal (as will be demonstrated).

Sedimentation

Sedimentation theory is based on removing all particles down to a certain size from the water before it reaches the berm. The particles only have to reach below the overflow berm elevation to be considered settled. This approach was used in the derivation of Equation 3.3. As will be shown, equation 3.3 actually represents the classical type 1 settling calculation.

A derivation of Equation 3.3 is as follows :

It takes a certain time t for a particle to move the length of the forebay L

$L = v_x t$

L is the forebay length, v_x is the horizontal velocity and t is time. During this same time t, a particle will fall a distance d (v_s is the settling velocity for that particle size).

$d = v_s t$

If d is set equal to the active storage depth above the forebay berm for a given design storm, then v_s represents the settling velocity of the largest particle size that will be settled in the forebay. Based on the design storm, an assumption can be made that the horizontal velocity through the forebay will be dictated by the cross-sectional area in the forebay (above the berm) and the outflow rate from the entire pond.

$v_x = Q/(dW)$

W is assumed representative of the width of flow, and Q represents the flow from the pond at depth d (relative to the berm height). Q is representative of the peak flow out of the pond during the design event (25 mm, etc.).

Substituting for v_x

$L/t = Q/(dW)$

Substituting for t based on the settling velocity

$L/(d/v_s) = Q/(dW)$

Rearranging

$L = Q/(Wv_s)$

Given a length to width ratio r

$L = rW$ or $W = L/r$

Substituting

$L^2 = rQ/v_s$

or $L = (rQ/v_s)^{0.5}$

This finding is the same as that provided by settling theory

$v_c = Q/A$

where A is the surface area of the forebay and v_c is equated to v_s

$A = LW$ or $A = L^2/r$

therefore

$v_s = Qr/L^2$

or $L = (rQ/v_s)^{0.5}$

Equation 3.3 can quickly be used to determine an appropriate forebay length based on a settling velocity and outflow from the pond. The length to width ratio should be equal to, or greater than, 2:1. Estimations of settling velocities for various particles sizes are generally based on

Stokes' Law or Newton's Law depending on the Reynolds Number (Stokes' law is only applicable for Reynolds Numbers <1 (ideally < 0.3).

Monitoring that was done as part of the National Urban Runoff Study in the U.S. (EPA, 1986), however, suggests that the settling velocities for particles in stormwater are much less than that given by Stokes' Law or Newton's Law. The settling velocities given by the NURP study are $1/100^{th}$ of that given by Stokes' Law. This can be explained in part by non-ideal settling characteristics and the fact that actual particles are not spherical in shape.

The relationship between settling velocity and particle size must be recognized in the design of the forebay. It is not sufficient to say that the forebay is sized to settle 150 µm unless the assumed settling velocity is also reported. For example, a forebay sized for 150 µm removal based on the NURP data would mean removal down to 20 µm based on Stokes' Law.

Another question is the distribution of suspended solids in stormwater. The NURP data indicates that 30% by mass is less than 40 µm. Based on the settling velocities associated with the particle sizes of the NURP data one could say that based on Stokes' Law, 80% of the suspended solids in stormwater by mass are less than 40 µm. A monitoring study in Ontario (Metropolitan Toronto, 1992) suggests that the particle size distribution is oriented towards the finer material (< 40 µm).

Although there is concern about the sediment distribution, the modelling which was undertaken in this study was based on the settling velocities from the NURP data. The results indicate that natural system stormwater management controls can achieve a high degree of solids removal even when the settling velocities are quite small.

One remaining factor should also be kept in mind. The calculations above (modelling, forebay sizing, Stokes' Law) are premised on discrete particle settling. There is evidence (personal comm. Dr. Krishnappen - CCIW) to suggest that flocculation and coagulation result in higher removals of suspended solids than expected based on discrete particle size estimations. This is confirmed by sampling studies (Stormceptor, 1994) for various stormwater devices which indicate that they are removing finer sediment than expected. There are two main mechanisms that are thought to infuence the flocculation of particles :

- physical activity of larger particles colliding with smaller particles, making them settle faster
- bacterial activity (bacterial will attach to fine sediment and form an adhesive which will bond to other fine sediment)

These mechanisms, would result in the estimations of settling based on discrete particle sizes being conservative.

Given the variability of suspended solids distribution in stormwater, settling velocities for various particle sizes, and the physical/biological activity occurring during settling, the use of

the NURP data is considered conservative and is recommended for use.

Scouring

Scouring is the physical activity of water scouring material from the bottom and sides of the forebay, making it available for transport downstream. The most common calculations which are made to determine scour potential relate to work than has been done for channel erosion (Camp, Hjulstrom, 1939). Although this may be applicable for the overall movement of water through the forebay, scouring may also occur from the jet action of water near the inlet of the forebay.

Simple calculations for jet scouring could not be found although work has been done in this area of hydraulics (Laursen, 1953). It is anticipated that resuspension or scour may be unavoidable at the inlet since a point discharge is being introduced into the forebay (ie. there will not be an uniform velocity in the forebay near the inlet). However, the use of the dispersion equations ensures that the jet effect from the inlet pipe will only affect the forebay area. Therefore, the approach was taken that the dispersion of a jet fluid should be investigated resulting in the adoption of Equations 3.4 and 3.5.

The use of Equations 3.4 and 3.5 with a minimum length to width ratio in the forebay of 2:1 and a 2 m depth of water in the forebay produces overall forebay velocities < 0.15 m/s. This velocity (0.15 m/s) is empirically denoted as the maximum velocity before scouring will occur in channels (Figure 4.6). A check should be made with the entire forebay cross-section to ensure that the velocities in the forebay are less than or equal to 0.15 m/s based on the inflow discharge.

APPENDIX H

RATIONAL METHOD - 25 mm INTENSITY

Rational Method

Several simulations using URBHYD (P'ng, 1982) in OTTHYMO were made to determine a relationship between the peak runoff as given by OTTHYMO for a 25 mm storm and the peak runoff as given by the Rational Method. Two sets of simulations were made with varying development areas (10 ha to 100 ha) and two levels of imperviousness (35% and 85%).

The flows from this simulation were used to back-calculate a value of I (rainfall intensity mm/h) for each simulation. A linear regression of the resultant intensities was performed which indicated that an intensity of 25 mm/h was reasonable for 35% imperviousness whereas 40 mm/h was reasonable for 85% imperviousness.

Using simple algebra :

$25 = 0.35\ x + b$
$40 = 0.85\ x + b$

$x = 30$ and $b = 14.5$

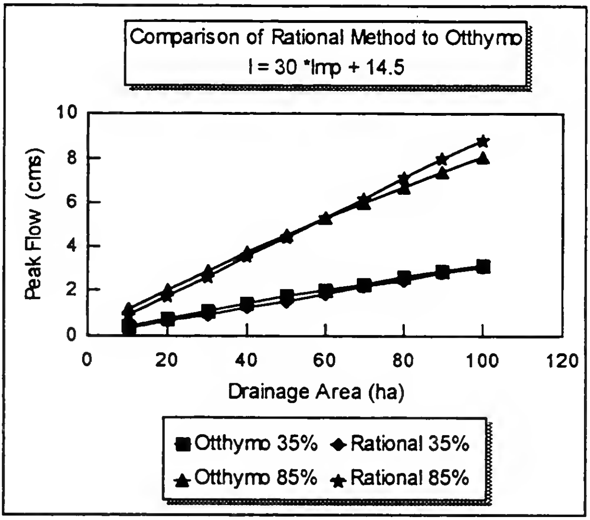
Therefore $I = 30\ Imp + 14.5$

Similarly, using Rational Coefficients instead of imperviousness and simple algebra

$I = 43\ C + 5.9$

where I is in mm/h

These equations were used to derive the flows using the Rational Method for the areas and levels imperviousness simulated with OTTHYMO. A comparison of the results using the Intensity Equation (Equation 3.7) is shown in Figure H.1.



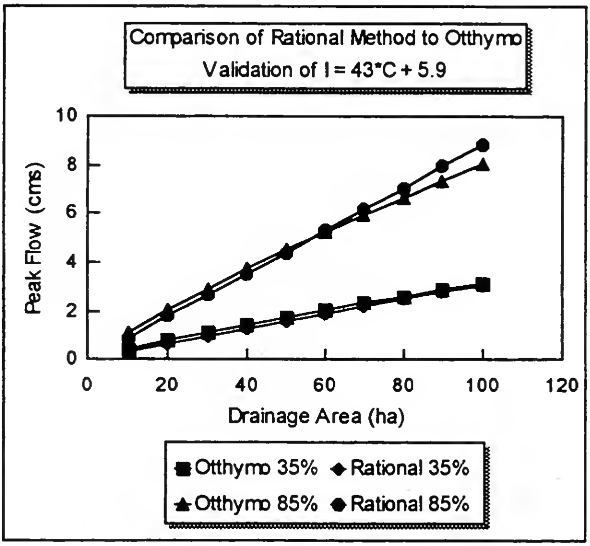


Figure H.1 Validation of Intensity Equation (Otthymo compared to Rational Method)

APPENDIX I

CONTINUOUS SIMULATION (SWMM + POND)

Water Quality Continuous Simulation

Modelling Methodology

SWMM models were developed for 100 ha of land with four levels of imperviousness (35%, 55%, 70%, and 85%). The SWMM models were simulated using Toronto International Airport meteorological data for a 20 year period of record (1967 - 1986). An hourly timestep was used for the simulations.

The resulting output from the SWMM model (hourly flows and suspended solids loadings) were input to a sedimentation model developed for the analysis of natural system stormwater controls (POND model). Four different types of end-of-pipe stormwater management facilities were modelled :

- wet ponds
- dry ponds (continuous and batch operation)
- wetlands
- infiltration systems

The POND model calculated the sediment removal efficiency of the pond on a real-time basis and provided a summary of the overall suspended solids removal efficiency for the 20 year simulation period.

Simulations were also conducted for various meteorological stations across the province with a wet pond and two storage volume sizes to assess the variability in removal performance with location across the province. The results from this simulation are presented in Figure 4.1 and indicate that there are negligible regional differences in suspended solids removal across Ontario.

SWMM Model

The SWMM model was selected to derive the suspended solids loadings to the stormwater management facilities for the following reasons :

- Continuous water quality and quantity simulation
- Snow accumulation/melt and snowpack redistribution routine
- Pollutant build-up and wash off for six parameters
- Accommodation of various hydraulic conveyance, storage and diversions elements within a drainage network
- Widespread acceptance of SWMM for continuous water quality and quantity modelling

SWMM 4.04 Program Development and Structure

SWMM was developed for the U.S. Environmental Protection Agency (US-EPA) in 1969 to 1971 by the following contractors: Metcalf and Eddy, Inc., University of Florida and Water Resources Engineers Inc. The model has been continually updated and modified by the original contractors and users of the program.

SWMM 4 predicts quantity and quality values for a specific catchment based on meteorological inputs and the physical attributes of the system such as size, conveyance elements, and storage/treatment units. The program is constructed of individual modules which simulate different aspects of the hydrological cycle.

The following program modules were used for this study:

- RAIN — The RAIN module processes long-term Atmospheric Environment Service (AES) rainfall data for input into the RUNOFF module.
- TEMP — In a similar manner, TEMP processes AES temperature data for input into RUNOFF for snow melt calculations.
- RUNOFF — RUNOFF generates surface and subsurface runoff based on the input precipitation time series (including snowfall), antecedent moisture conditions, and subcatchment description. Pollutant constituents may also be generated and subsequently washed off creating 'end of the pipe' pollutographs.
- TRANSPORT — The TRANSPORT module routes flows and pollutants through the sewer and/or drainage system. Dry weather flow and infiltration into the system may also be simulated.

Preliminary verification of the SWMM 4.04 code indicated that various errors, such as the metric conversion in the TEMP module, were present in the original code. These errors were subsequently corrected and the affected subroutines re-compiled using Microsoft Fortran 5.1.

Meterological Data

SWMM 4.04 is a comprehensive hydrological simulation program, and as such, requires an extensive amount of input data. In order to simulate both rainfall and snow accumulation/melt, the following meterological data were required:

- Precipitation
- Air temperature
- Wind velocity

- Evaporation rates

Precipitation

The precipitation time series data from Toronto Pearson International Airport weather station (6158733) between 1967 and 1986 was used for this study.

AES records of hourly rainfall do not include snowfall. Therefore a FORTRAN program (AESWMM), developed by Marshall Macklin Monaghan, was used to compare the AES daily precipitation values (which include snowfall) to the hourly records and generate the missing hourly snowfall and rainfall values.

Temperature

To simulate snow fall and snow melt, SWMM 4.04 requires daily minimum and maximum air temperatures. Hourly dry bulb temperatures for the airport gauge were obtained for the period 1965 to 1989. Marshall Macklin Monaghan developed a Fortran program (AESHRTEM) to calculate the daily minimum and maximum temperatures from the hourly information.

Evapotranspiration

Pan evaporation rates were obtained for the period 1951 to 1980 from the Canadian Climatic Normals Volume 9. Records from the Guelph Ontario Agriculture College (O.A.C.) station were used since they are the most extensive and complete evaporation records for southern Ontario.

Pan evaporation represents the evaporative losses from water bodies such as lakes and small reservoirs. Recognizing that the highway surface area in the study area has negligible depression storage, it was assumed that negligible water would be retained on the pavement and that evaporation would primarily occur from water held on the R.O.W.s. Pan evaporation data was corrected to reflect the lower degree of exposure and reflectance of vegetative surfaces in comparison to bodies of water. Monthly correction factors (Saxten et al, 1974) were used to adjust the Pan evaporation rates to reflect potential evapotranspiration (PET). Table I.1 lists the evaporation rates calculated for the months from April to October. Negligible evaporation was assumed throughout the winter months.

Wind Velocities

Wind speeds are required to simulate potential snow melt. Mean wind velocities were obtained for the Toronto Scarborough College from the Canadian Climatic Normals, Volume 5, 1951 - 1980 (Table I.2) Only values for months with potential snow melt

were included in the model.

Table I.1 Evaporation and Evapotranspiration Rates			
Month	Pan* (mm/day)	PET/Pan Ratio	PET (mm/day)
April	2.59	0.84	2.18
May	3.95	0.88	3.48
June	4.61	0.88	4.06
July	4.74	0.88	4.17
August	3.81	0.86	3.28
September	2.61	0.80	2.09
October	1.55	0.70	1.09

*Daily means for Guelph O.A.C. (1951 -1980).

Table I.2 Monthly Mean Wind Velocities												
Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
Mean Wind Velocity (km/h)*	12.1	11.6	12.5	12.5	9.7	8.2	8.3	7.2	7.4	8.6	10.2	11.0

* All directions

Model Discretization

One 100 ha catchment was modelled.

Catchment Width

The width of a catchment, as input into SWMM, is the physical width of overland flow. Catchment width affects the time of concentration or the time for a wave of water to travel from the remote point of catchment to the outlet.

travel from the remote point of catchment to the outlet.

A catchment width of 850 m was used for the 100 ha drainage area in this study.

Depression Storage

Depression (retention) storage is the volume that must be filled prior to the occurrence of runoff on both pervious and impervious areas. In the RUNOFF module it represents an initial abstraction subject to infiltration (pervious areas) and evaporation (impervious and pervious areas).

The impervious areas were assumed to have a depression storage of 0.5 mm whereas the pervious areas were modelled with 4.67 mm of depression storage (these are standard numbers used in event simulation with the OTTHYMO model (Wisner and P'ng, 1983)).

Infiltration

Infiltration from pervious areas was computed using Horton's equation.

$$f_p = f_c + (f_o - f_c)\, e^{-kt}$$ Equation I.1 Horton's Infiltration

where:

f_p = infiltration capacity of the soil (mm/h)
f_c = minimum or ultimate infiltration capacity (10.5mm/h)
f_o = maximum or initial infiltration capacity (63.5mm/h)
t = time from beginning of storm (sec)
k = decay coefficient (0.00115 sec^{-1})

The infiltration parameters used were in accordance with those which are typically used for event modelling with the OTTHYMO model in Ontario.

Snowmelt Data

Snowmelt is an important source of surface runoff in the context of long term continuous simulation. Although the melt flows are typically low for urban catchments, they may be sustained over several days and remove a significant fraction of pollutants deposited during the winter months (Huber and Dickinson, 1988).

In the RUNOFF module, hourly precipitation data are utilized along with the daily maximum and minimum temperatures to generate precipitation as snowfall. Precipitation as snowfall is only generated at temperatures below a user specified dividing temperature. A dividing temperature of 0.0 °C was used in this study. A thorough explanation of snow generation and snowmelt routines are contained in the SWMM 4.04 User's Manual.

Gauge Catch Deficiency Correction (SCF)

Snow gauges tend to underestimate the actual depth of snow which has fallen. The deficiency varies according to the type of gauge (whether or not the gauge is shielded), the location of the gauge (degree of open exposure/tree cover), and the wind velocity during periods of snowfall (Huber and Dickinson, 1988). The SCF is used to account for the underestimation of gauge recordings and to calibrate the simulation of snowfall.

According to the Canadian Climatic Normals for the Toronto Scarborough Station (1951 - 1980) approximately 17% of yearly precipitation results in snowfall. Preliminary modelling indicated that the SWMM generated precipitation as percent snowfall was consistently less than the expected average yearly percentage snowfall for this area.

Anderson (1973) developed a relationship between wind speed and the catch deficiency correction factor (SCF) for non-shielded and alter shielded snow gauges which is graphically represented in the SWMM User's Manual. AES uses a shielded Nipher gauge to record snowfall.

The average wind speed for the Greater Toronto Area (GTA) during the winter months is approximately 11.56 km/h. Using Anderson's relationship a SCF value of 1.15 was applied to the model.

Free Water Holding Capacity (FWFRAC)

During the SWMM melt routines, runoff does not necessarily coincide with snowmelt. Since a snowpack is a porous medium it has the potential to temporarily store melt water, which is termed the free water holding capacity of a snowpack. FWFRAC is equal to the free water holding capacity as a fraction of snowpack depth. SWMM simply treats FWFRAC as a linear reservoir and routes the liquid runoff accordingly, which causes a minor delay and attenuation of the runoff during the melt. FWFRAC varies according to condition of the snowpack. Huber and Dickinson (1988) have documented various FWFRAC values for different snowpack conditions.

The model assumed that snowfall on pervious surfaces will exhibit a typical deep snowpack (> 25.4 cm) and therefore was assigned a value of 0.05. The snowfall remaining on the impervious surface was assumed to exhibit a shallow winter snowpack condition, which ranges in FWFRAC between 0.05 to 0.25. A value of 0.2 was selected to model the snowpack on the impervious areas.

Snow Redistribution

SWMM 4.04 is capable of modelling different snow removal and redistribution strategies (eg. ploughing) which are typical of practices in urban areas. SWMM assumes all snow subject to "redistribution" resides on "normally bare" surfaces such as streets, sidewalks

and parking lots. Five types of redistribution are available to closely model the management practices characteristic of urban areas.

It was assumed, in this study, that the snow from normally bare impervious areas was redistributed; half being transferred to snow covered impervious areas and the other half to snow covered pervious areas.

Each subcatchment requires a depth of snow (WEPLOW) as input, above which redistribution occurs. In this study, WEPLOW was assumed to be 4 cm.

It should be noted, however, that the SWMM snowfall redistribution computations do not transfer pollutants with the snow and do not affect their build-up and/or wash-off functions.

Areal Extent of Snow Cover

SWMM 4.04 allows the user to specify the way in which a snowpack melts. This allows representation of the effects of shading, drifting, topography, and redistribution. Certain portions of a subcatchment will become bare before others, and the fraction of area covered by snow will vary with time. The relationship between the fraction of area covered by snow (SNC) and the amount of snow on the catchment at any given time is called an Areal Depletion Curve (ADC). Five different depletion curves are available in SWMM.

In this study, typical ADCs for pervious and impervious areas were taken from the SWMM 4.04 User's Manual.

Pollutant Data

Pollutant Build-up

Atmospheric deposition of pollutants occurs during both dry and wet weather conditions. SWMM is capable of modelling the wet depositional process by allowing the user to specify a constant pollutant concentration for precipitation. Dry deposition occurring as a result of pollutant transport between sub-catchments via wind vectors is not possible in SWMM. As a result, no redistribution of pollutants between subcatchments, except due to wash off, was modelled.

Another factor affecting pollutant deposition is the number of dry days preceding rainfall events. The longer the dry day interval, the greater the pollutant build-up, and hence, the larger the stormwater pollutant loading. Although pollutants will tend to be redistributed as the dry day interval increases, the model assumes that no redistribution of pollutants outside of the study area will occur. Therefore, the pollutant concentration

will be higher for storms with longer antecedent dry days.

Pollutant Build-up Formulations

SWMM 4 models four pollutant build-up techniques :

- Linear
- Power-Linear
- Exponential
- Michealis-Menton

Although most pollutant data suggests linear build-up (e.g. kg/100 m curb-day; kg/2 lane-km) there is no consensus as to whether build-up is linear or asymptotic in behaviour.

A power linear formulation was chosen so that the rate of build-up could be adjusted to reflect a rapid build-up observed in monitored data from other sources.

Equation I.2 represents the power-linear build-up.

$$PSHED = QFACT_3 \times t^{(QFACT_2)} \qquad \text{Equation I.2 Power Linear}$$

where:

PSHED = Constituent Quantity (kg); $PSHED \leq QFACT_1$
$QFACT_3$ = Build-up rate (kg/km-Curb·day)
$QFACT_2$ = Exponent (user defined)
$QFACT_1$ = Upper Limit (kg)
t = Time (days)

Build-up Parameter Estimation

Only total suspended solids was selected for modelling. Since other pollutants (ie. metals, nutrients, bacteria, oils) have an affinity for suspended solids, the use of this parameter was assumed to be indicative of the relative removal for the other pollutants.

The power-linear build-up function in SWMM requires the user to specify a build-up rate, an exponent and an maximum quantity limit (optional) for the suspended solids loading.

Build-up Rate

Table I.3 summarizes the pollutant loading rates used as input into SWMM.

Build-up Limit and Exponent

The limit of build-up and the power parameters were first estimated based on values referenced in the SWMM manual, then subsequently adjusted based on initial modelling results until the RUNOFF mean pollutant concentrations were within the range of 150 to 600 mg/l. The resultant build-up limit and exponent used were 1000 kg/km and 0.72, respectively.

Precipitation Quality

Numerous pollutant constituents, including organics, metals, solids and nutrients have been found in rainfall. Although the contribution of rainfall to the loading of most constituents may be small relative to surface sources, rainfall has been shown to be an important contributor of some nutrients (Huber and Dickinson, 1988). The pollutant concentrations attributed to rainfall were obtained from the USFHA monitoring results for the Milwaukee site (M.M. Dillon, 1990) and are listed in Table I.3.

Table I.3 SWMM Pollution Build-up

Pollutant	Surface Loading $(kg/km \cdot Day)^{0.72}$			
	35% Impervious	55% Impervious	70% Impervious	85% Impervious
TSS	600	1800	2840	3960

Pollutant Wash-off

Wash-off is the process of pollutant scour and transport from the subcatchment surface during storm events. The magnitude of pollutant wash-off from impervious surfaces is a function of:

- The amount (build-up) of constituent available to be washed off
- The runoff rate (related to the kinetic energy of the rainfall)

SWMM models the conveyance of pollutants using sediment transport theory since many of the constituents are in particulate form. The conveyance of pollutants is assumed proportional to the runoff rate raised to some power. Equation I.3 denotes the SWMM wash-off formula.

$$POFF(t) = RCOEF/3600 \times r^{WASHPO} \times PSHED$$ Equation I.3 Pollutant Wash-off

where:

POFF = Constituent load washed off at time, t, (kg/sec)
PSHED = Quantity of constituent available for washoff at time, t, (kg)
RCOEF = Washoff coefficient (mm^{-1})
WASHPO = Washoff power
r = Runoff rate (mm/h)

SWMM requires values for RCOEF and WASHPO as input parameters to the model. These values were estimated from the SWMM users manual.

Storage and Conveyance Elements

A continuous series of flows and pollutant concentrations is required for input to the POND program. In order to have a continuous series of flows and pollutant concentrations, a dummy pipe was used in the TRANSPORT block. This dummy pipe was established since the RUNOFF block outputs flow and pollutant concentrations only when there is a runoff event. Routing the RUNOFF block output through this dummy pipe filled the inter-event timestep flow and pollutant concentrations with zeros.

POND Continuous Analysis Program

Most empirical water quality assessment methods are based on either a precipitation volume (event) or an assessment of long term precipitation indices (averages and coefficients of variation) and facility parameters (storage volume or retention time). These methodologies do not account for the variability and seasonality of stormwater runoff. In order to adequately assess suspended solids removal, calculations must be based on the volume of storm water present, the concentration of suspended solids in the pond, the inflow of stormwater to the pond, and outflow of water from the pond at any one given time. Therefore, continuous modelling of settling in the pond was performed since it allowed a more realistic prediction of expected facility effectiveness under actual operating conditions. The POND program was developed to read in real time values of flow and sediment loading and perform calculations of settling and/or conveyance through a stormwater quality pond.

Suspended Solids Settling Characteristics

The modelling of the pond on a real time basis requires a definition of the settling behaviour of suspended particles on a real time basis. The settling characteristics of solids is affected by the flow conditions in the pond. Under quiescent conditions turbulence is minimized and settling is enhanced. Under dynamic conditions the opposite is true. In this study, dynamic conditions were defined to occur when the influent flow volume to the pond for a given time step was equal

to, or greater than, 1% of the water contained in the pond at that particular time step.

Quiescent settling was approximated using the settling velocity, the time step, and the depth of the pond at the time in question (Equation I.4). Completely mixed conditions were assumed.

$$R_q = \frac{v_s \Delta t}{d}$$ Equation I.4 Quiescent Settling

where :

R_q	=	TSS removal under quiescent conditions
v_s	=	settling velocity of particulate matter (m/s)
d	=	depth of pond (m)
Δt	=	time step (s)

Dynamic settling was approximated using the method presented by Fair and Geyer which is the standard methodology approved by the U.S. EPA for detention basin analysis (USEPA, 1986). This methodology has also been used in Canada (Wisner, 1990; Marshall Macklin Monaghan Limited, 1991). The equation used to define dynamic settling is presented in Equation I.5.

$$R_d = 1 - \left[1 + \frac{1}{n} \cdot \frac{v_s}{Q/A} \right]^{-n}$$ Equation I.5 Dynamic Settling

where:

R_d	=	TSS removal under dynamic conditions (m^3/m^3)
Q	=	average of inflow and outflow (m^3/s)
A	=	surface area of pond (m^2)
v_s	=	settling velocity of particulate matter (m/s)
n	=	turbulence factor

The turbulence factor provides an indication of the design of the pond. If the pond is poorly designed (eg. outlet next to inlet, large width to length ratio) then n will decrease (greater turbulence). As n approaches infinity ideal settling characteristics are achieved. N was set equal to 3 in this analysis (which represents average or good settling characteristics) when the influent volume for a 20 minute period was equal to, or greater than, 5% of the volume of water in the pond at that time step. An N value of 5 was used for near quiescent conditions (1% of pond volume < influent volume < 5% of pond volume).

Equation I.5 predicts a long term average removal rate but is not suited for real time calculations. The equation is based on a constant flowrate (Q) which is related to the average retention time of particles in the pond (T) :

$$T = \frac{V}{Q}$$ Equation I.6 Retention Time

where :

T = retention time in the pond
Q = constant flowrate into/out of pond
V = volume of water in pond

This relationship (Equation I.6) was used to modify Equation I.5 for use on a real time basis as below :

$$R_d = [\![1 - [1 + \frac{1}{n} \cdot \frac{v_s}{Q_t/A}]^{-n}]\!] \frac{\Delta t}{T_t}$$ Equation I.7 Real Time Dynamic Settling

where :

R_d = TSS removal under dynamic conditions
Q_t = average of inflow and outflow for time step Δt (m^3/s)
A = surface area of pond (m^2)
v_s = settling velocity of particulate matter (m/s)
n = turbulence factor (set = 3)
Δt = time step (s)
T_t = retention time based on Q_t and volume in pond at time t (V_t)

In equation I.7, erroneous answers can occur if T_t is less than Δt. The factor $\Delta t/T_t$ was incorporated to account for T_t being much longer that Δt on a real time basis. Hence, $\Delta t/T_t$ is not allowed to be greater than 1.

Equations I.4 and I.7 were used to formulate a continuous model of settling rates for particles during quiescent and dynamic conditions respectively. The definition of Q_t implies that the pond is being treated as a completely mixed facility. The concentration of suspended solids in the pond was assumed homogeneous and was based on a simple accounting procedure of how much came in, how much settled out, and how much was discharged in the effluent.

The settling velocity is related to the size of particle. As such, the use of a single settling

velocity would not be representative of suspended solids in stormwater since there is a distribution of particle sizes, each of which having different settling velocities. Research on particle size distribution in stormwater indicates that the distribution can be approximated by 5 different size classes. Table I.4 indicates the standard size classes used by the US EPA methodology for detention basin analysis.

Table I.4 Particle Size Distribution in Storm Water

Size Fraction	% of Particle Mass	Average v_s (m/s) ≤
1	0 - 20	0.00000254
2	20 - 40	0.00002540
3	40 - 60	0.00012700
4	60 - 80	0.00059267
5	80 - 100	0.00550333

Table I.4 was revised to include additional Canadian research on the distribution of particle sizes in storm water.

Table I.5 Revised Particle Size Distribution in Storm Water

Size Fraction	% of Particle Mass	Average v_s (m/s) ≤
1	0 - 20	0.00000254
2	20 - 30	0.0000130 (40 μm)
3	30 - 40	0.00002540
4	40 - 60	0.00012700
5	60 - 80	0.00059267
6	80 - 100	0.00550333

Table I.5 indicates that 30 % of the suspended solids in storm water are smaller than the 40 µm size. Therefore, an extended detention dry pond could only remove approximately 70 % of the suspended solids in storm water unless it had a very large surface area compared to the storage depth. For example, the settling velocity associated with size fraction 1 translates to a depth of fall of 1 m in about 110 hours.

It should be noted that the settling velocities associated with the particle sizes in Table 2 are

extremely small when compared to Stokes' Law (100 times smaller). If the particle sizes were calculated from Stokes' Law based on the settling velocities given in Table 2, 80% of the suspended solids in stormwater would be smaller than 40 μm. The settling velocities in Table 2 are from monitored data and are likely conservative, given that flocculation settling may also be occurring in stormwater facilities.

There is considerable debate regarding the true particle size distribution and settling velocities for stormwater suspended solids. As with all stormwater related variables the particle size and settling velocity will depend on numerous factors, many of which will be site specific. As such, the actual distribution of particle sizes for the site, and settling characteristics of a stormwater facility, will govern the operational removal efficiency of the facility.

For the purposes of modelling, the influent suspended solids loading was divided into the 6 size fractions at the inlet of the pond. Each size fraction was accounted for independently from the other fractions. Six settling calculations were made at each time step to determine the respective removal rates for each of the particle size classifications.

Literature values often report up to 90% removal of suspended solids after 24 hours retention. Removal values this high can only be attained through mixing and removal in the permanent pool of a wet pond. Stormwater mixes with the permanent pool when it enters into the pond such that any effluent is diluted. Any sediment which remains in the permanent pool after a storm has subsided has the potential to be removed under quiescent settling conditions before the occurrence of the next storm.

The POND model does not account for numerous factors such as wind, density changes, filtering of sediments by vegetation, etc. In reality the reaction of sediment in a pond is a function of many parameters and includes re-suspension as well as sedimentation. These secondary factors (wind, density changes, vegetation impacts) have been qualitatively taken into account through design guidance provided in Chapter 3 of the manual.

POND Input

The POND model requires the following information:

- **Continuous Time Series of Flow and Pollutant Loads**: output from SWMM
- **Active Storage Parameters**: active storage volume and depth
- **Permanent Pool Volume and Depth**: permanent pool volume and depth
- **Retention Time**: The drawdown time over which the maximum active storage volume is released.
- **Average Daily Baseflow**: The average daily baseflow is the value POND uses to determine when a storm event occurs. The average baseflow was set equal to 0 for this study
- **Inlet Pipe Capacity (if required)**: Flows exceeding the capacity of the inlet pipe to the

pond, bypass the pond. The inlet pipe capacity was set equal to the peak flow from a 2 hour 25 mm storm (for the first flush simulation only).

- **Batch or Continuous Flow :** Batch file assumed mechanical/electrical outlet control whereas continuous flow assumed a hydraulic outlet
- **First Flush (if required) :** First flush flows would divert water and contaminants upstream of the facility if the facility was full. Non first flush facilities would mix contents of incoming water with the facility and overflow from the facility itself
- **Internal calculation timestep :** The internal timestep can be chosen to be any number less than the SWMM timestep. A timestep of 10 minutes was used for this study.
- **Summer or Entire Year :** The POND program can simulate for the entire year or just the summer periods. The entire year was simulated for this study.

POND Output

POND outputs the following summary information per event and for the entire period of record:

- the total runoff volume
- the total overflow volume
- the total TSS load into the pond
- the total TSS load in the pond outflow
- the total TSS load in the overflow (bypass)
- the number of bypasses
- the pond efficiency (% TSS removal)
- the distribution of effluent TSS (overflow/bypass and outflow) by size category

Stormwater Management Facilities

All of the storage volumes provided in Table 4.1 and Figures 4.2 through 4.5 are based on specific depths for both the permanent pool and the active storage. The depths greatly affect storage requirements as is demonstrated by a comparison of the storage requirements between wet ponds and wetlands. The depths which were used in the simulations were :

- dry ponds (maximum depth of 2 m)
- wet ponds (permanent pool depth = 2 m, active storage max. depth = 2m)
- wetlands (permanent pool depth = 0.3 m, active storage max. depth = 1m)
- infiltration (depth = 1.5 m)

The extended detention (ED) wet facilities (wetland, wet pond) had a constant extended detention storage of 40 m^3/ha while the permanent pool volume was allowed to vary. Earlier technical simulations indicated that the permanent pool volume is more efficient than extended detention storage for TSS removal. This result is shown by comparing Figure I.1 with Figure I.2.

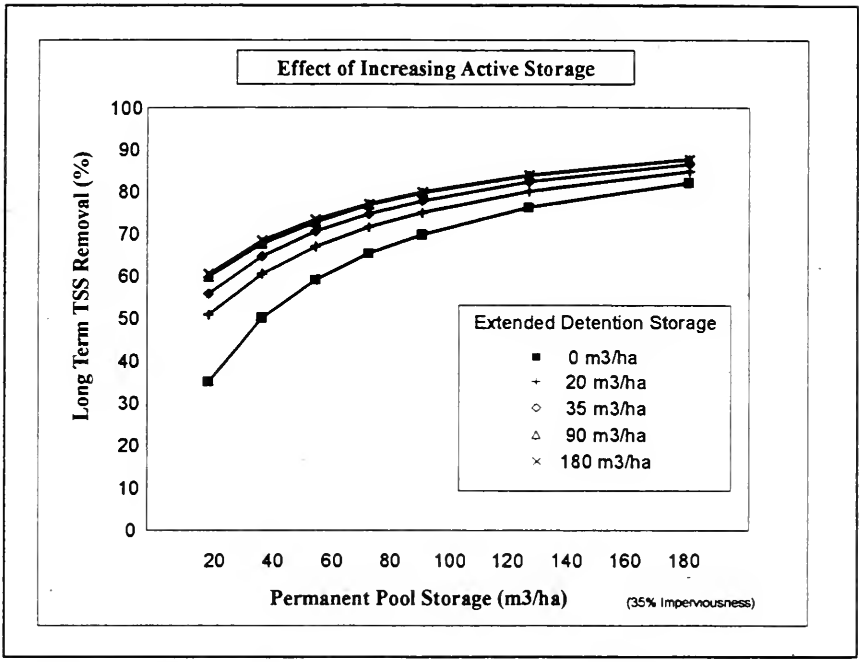


Figure I.1 Marginal Effectiveness of Extended Detention Storage

Drawdown Time

All of the simulations were based on a 24 hour drawdown of the active storage (extended detention) provided in the facility.

First Flush / Combined Facilities

All of the stormwater facilities were modelled as combined facilities. Stormwater which overflowed the facility due to insufficient active storage was allowed to mix with the facility contents (ie. completely mixed conditions). This represents an overflow near the outlet of the facility. A simulation was made comparing a facility which was modelled as a first flush facility (by-pass occurred either when the inflow rate was higher than the capacity of the first flush pipe [the capacity of the first flush pipe was set equal to the peak flow from a 25 mm storm], or when the facility reached it's design capacity) with the same facility modelled as a combined facility.

The results from the modelling indicated that there was negligible difference between the performance of the two facilities. These results, however, reflect the assumptions in the modelling of :

- no re-suspension/scour of material which had previously settled.
- completely mixed conditions (ie. good pond design without dead zones)

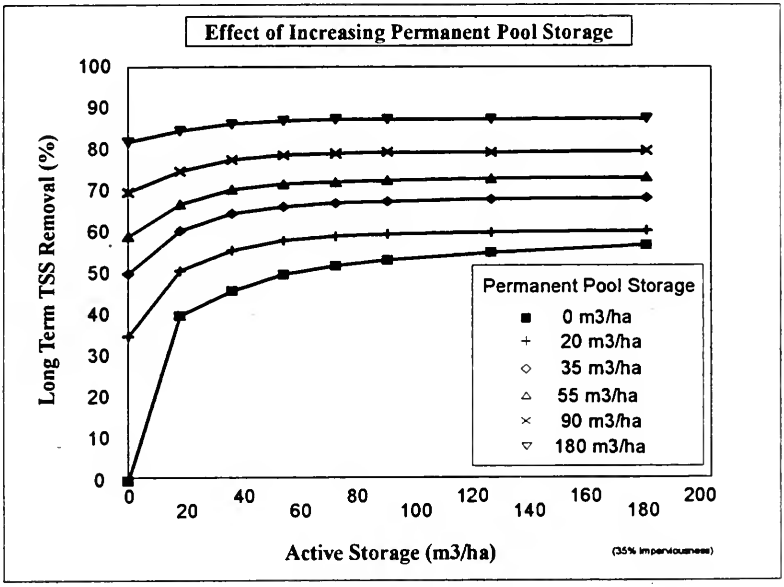


Figure I.2 Marginal Effectiveness of Permanent Pool Storage

Infiltration

The POND program is a sedimentation model. For traditional sedimentation facilities (dry ponds, wet ponds, wetlands), the POND model will directly calculate the removal efficiency. In infiltration or filtration facilities, however, one can argue that any water which is influent to the facility is treated. This assumption was made, and translated into the following rules for assessing infiltration facilities :

- for the simulations with 35% and 55% imperviousness, a removal

efficiency of 95% was assumed for water which was infiltrated/filtrated

- for 70% and 85% imperviousness, a removal efficiency of 90% was assumed for water which was infiltrated/filtrated

Dry Pond (Batch Operation)

The dry pond was simulated in two modes of operation :

- continuous
- batch

Continuous operation represents an outlet which operates based on the simple hydraulics of water depth in the pond (ie. a perforated riser or reverse-sloped pipe). Batch operation represents an outlet which is electrically/mechanically closed such that :

- the outlet is closed at the beginning of a stormwater runoff event
- the outlet is opened after the desired detention time and allowed to drain a specific rate.

In the batch simulations, a 24 hour detention time was used. A drawdown time of 12 hours was used to drain the contents of the pond after the detention time.

APPENDIX J

STATISTICAL WATER QUALITY POND MODEL

1. Principles of the Statistical Approach

1.1 Derived Probability Distribution Theory

The basis of the statistical approach to modelling stormwater detention pond performance is derived probability distribution theory. The theory states that the probability distribution of a dependent random variable is fundamentally related to and may be derived from those of the independent random variables using the functional relationship between the dependent and independent variables. The success of such a derivation is dependent upon the determination and integration of the joint probability density function of the independent variables. (For details of derivations, see Adams and Bontje [1983] and Benjamin and Cornell [1970].)

1.2 Statistical Representation of Rainfall Series

A continuous rainfall series can be separated into individual rainfall events according to a selected interevent time definition (IETD). The IETD is defined as the minimum elapsed time with no rainfall that separates rainfall events. Thus, rainfall pulses separated by a time greater than the IETD are considered to be different events.

The separated rainfall events then can be characterized by event rainfall volume, v(mm); event duration, t(hr) and interevent time, b(hr). Variations within a single event are usually not considered. Previous studies [e.g., Adams et al., 1986; Li, 1991] have shown that v,t, and b can be practically assumed to be mutually independent and are often exponentially distributed as follows:

- Event rainfall volume, v(mm)

$$f_V(v)=\zeta e^{-\zeta v}; \quad \zeta=\frac{1}{\bar{v}} \tag{1}$$

- Event duration, t(hr)

$$f_T(t)=\lambda e^{-\lambda t}; \quad \lambda=\frac{1}{\bar{t}} \tag{2}$$

- Interevent time, b(hr)

$$f_B(b)=\psi e^{-\psi b}; \quad \psi=\frac{1}{\bar{b}} \tag{3}$$

The values of parameters ζ, λ and ψ can be obtained by conducting storm event analyses on long term rainfall records.

1.3 Transformation of Rainfall To Runoff

The transformation of rainfall volume to runoff volume is modelled as follows:

$$v_r = \begin{cases} 0 & \text{if } v \le S_d \\ \phi(v - S_d) & \text{if } v > S_d \end{cases} \tag{4}$$

in which v_r is the event runoff volume (mm); S_d is the depression storage of the contributing catchment (mm); and ϕ is the runoff coefficient of the catchment. The duration of the runoff event is assumed to be the same as the corresponding rainfall event.

1.4 Quiescent Settling and Dynamic Settling

Due to the intermittent and random nature of urban runoff, the operating conditions of stormwater quality ponds change from quiescent, without any inflow and outflow, to dynamic with varying inflow and/or outflow. Herein, suspended solid settling in stormwater quality ponds is distinguished by quiescent settling and dynamic settling conditions. Quiescent settling describes particle settling in still water, while dynamic settling describes particle settling in flowing water. It is assumed that Chen's[1975] formulation of sedimentation efficiency is valid for the computation of dynamic settling in stormwater quality ponds. In Chen's formulation, sedimentation efficiency, E_d, is expressed as a function of surface loading rate for various turbulent flow conditions. The fully turbulent curve is approximated by

$$E_d = 1 - e^{-\frac{v_s A}{\Omega}} \tag{5}$$

where Ω is the pond discharge rate (m^3/hr), v_s is particle settling velocity (m/hr) and A is the pond surface area (m^2).

1.5 Definition of a Loading Cycle

The starting point in the statistical analysis of stormwater quality ponds is the definition of the loading cycle. Under a continuous, flow-through mode of operation, the pond is filled gradually during a runoff event, or if the runoff intensity is small, the pond may maintain a constant (or even declining) water level since the pond is drained through outlet control structures at the same time. After a runoff event, outflow from the pond continues until the pond is emptied or the pond water level drops to the permanent pool level. The next runoff event may occur before the pond is emptied (or while the water level is above the permanent pool level) or when the pond is empty (or at its permanent pool level).

The loading cycle of a stormwater quality pond is thus defined as the time period between the starting points of consecutive runoff events. The derivations of the statistical models in the following sections focus on the analysis of the probability distribution functions of pond performance within a loading cycle. The long term average performance of the pond is further derived from the derived probability distribution functions.

2. Statistical Analysis of Extended Detention Dry Ponds

Based on the principles outlined in the previous sections, assuming that the current runoff event begins with the pond being emptied, the possible time histories of a loading cycle of an extended detention dry pond can be summarized as follows:

(1) The runoff volume is large enough to fill the pond and cause a certain amount of spill; the following dry period is long enough so that the pond is emptied before the next runoff event.

(2) The runoff volume is large enough to fill the pond and cause a certain amount of spill, but the following dry period is so short that the pond is not empty when the next runoff event begins.

(3) The runoff volume is not large enough to fill the pond; and the following dry period is long enough to empty the partly filled pond before the next runoff event.

(4) The runoff volume is not large enough to fill the pond; and the following dry period is so short that the pond is not emptied when the next runoff event begins.

(5) The intensity of runoff is so small that the pond remains at zero stage and runoff flows through the pond without detention.

The mechanism of suspended solids removal in an extended detention dry pond under continuous, flow-through mode is dynamic settling. Assuming that the controlled outflow rate from the pond is a constant, denoted by Ω, and that the average water surface area is also constant, denoted by A, then the average suspended solids removal efficiency can be calculated by empirical equations such as Eq. 5. The dynamic sedimentation rate, D_d (mass/time), can be calculated as follows:

$$D_d = E_d \Omega C_0 \tag{6}$$

where C_0 is the long term average suspended solids concentration of the incoming runoff.

Assuming that spills from the emergency spillway receive zero treatment, within each loading cycle, the total mass of suspended solids removed is then the product of the average removal

efficiency E_d, the outflow rate Ω, the time during which this dynamic settling process takes place and the average suspended solid concentration of the incoming runoff. Corresponding to each of the five possible cases of the time history of a loading cycle, the suspended solid mass removed in a loading cycle, R_m, can then be expressed as in Eq. 7.

$$R_m=\begin{cases} D_d(t+\frac{S}{\Omega}) & \text{if } b>\frac{S}{\Omega}, \phi(v-S_d)-\Omega t>S \\ D_d(t+b) & \text{if } b\leq\frac{S}{\Omega}, \phi(v-S_d)-\Omega t\geq b\Omega \\ D_d[t-(t-\frac{\phi(v-S_d)}{\Omega})] & \text{if } \begin{cases} b>\frac{S}{\Omega}, 0<\phi(v-S_d)-\Omega t\leq S \\ b\leq\frac{S}{\Omega}, 0<\phi(v-S_d)-\Omega t<b\Omega \end{cases} \\ D_d(t-t) & \text{if } \phi(v-S_d)-\Omega t\leq 0 \end{cases} \tag{7}$$

In which S is the storage volume of the extended detention dry pond. Cases 2 and 4 have been combined into one case because they have the same expression for total mass removed in a loading cycle. The above expressions for R_m are also written in such a way that it is obvious that R_m is a combination of two parts. i.e.

$$R_m=D_d(t-T_d) \tag{8}$$

where

$$T_d=\begin{cases} -\frac{S}{\Omega} & \text{if } b>\frac{S}{\Omega}, \phi(v-S_d)-\Omega t>S \\ -b & \text{if } b\leq\frac{S}{\Omega}, \phi(v-S_d)-\Omega t\geq b\Omega \\ -[\frac{\phi(v-S_d)}{\Omega}-t] & \text{if } \begin{cases} b>\frac{S}{\Omega}, 0<\phi(v-S_d)-\Omega t\leq S \\ b\leq\frac{S}{\Omega}, 0<\phi(v-S_d)-\Omega t<b\Omega \end{cases} \\ t & \text{if } \phi(v-S_d)-\Omega t\leq 0 \end{cases} \tag{9}$$

According to the above definition of T_d, using derived probability distribution theory and

assuming that v, t, and b are exponentially distributed mutually independent random variables as described in Eqs. 1,2 and 3, the following results for the probability density function of T_d can be derived:

$$f(t_d)=\begin{cases} \dfrac{\lambda\phi}{\lambda\phi+\zeta\Omega}e^{-\psi\frac{S}{\Omega}}e^{-\psi S_d}e^{-\zeta\frac{S}{\phi}} & \text{when} \quad T_d=-\dfrac{S}{\Omega} \\ \dfrac{\lambda(\psi\phi+\zeta\Omega)}{\lambda\phi+\zeta\Omega}e^{-\zeta S_d}e^{(\psi+\zeta\frac{\Omega}{\phi})T_d} & \text{when} \quad -\dfrac{S}{\Omega}\leq T_d<0 \\ \lambda e^{-\lambda T_d}-\lambda e^{-\zeta S_d}e^{-(\lambda+\zeta\frac{\Omega}{\phi})T_d} & \text{when} \quad T_d>0 \end{cases} \tag{10}$$

The mean of T_d can subsequently be derived as follows:

$$\begin{aligned} \overline{T_d}=&C_1^2C_4^2[(\frac{S}{\Omega}+C_5)C_2C_3-C_5](1-C_2c_3) \\ &+(\frac{1}{\lambda}-\frac{1}{\lambda}C_1C_4^2)(1-C_1C_4)-\frac{S}{\Omega}C_1C_2C_3C_4 \end{aligned} \tag{11}$$

$$C_1=e^{-\zeta S_d} \tag{12}$$

$$C_2=e^{-\psi\frac{S}{\Omega}} \tag{13}$$

$$C_3=e^{-\zeta\frac{S}{\Omega}} \tag{14}$$

$$C_4=\frac{\lambda\phi}{\lambda\phi+\zeta\Omega} \tag{15}$$

Therefore, the mean mass of suspended solids removed in a loading cycle is

$$\overline{R_m} = D_d(\bar{t} + T_d) \tag{16}$$

$$C_S = \frac{\phi}{\psi\phi + \zeta\Omega} \tag{17}$$

By assuming a constant concentration C_0 of suspended solids with an average settling velocity V_s in all incoming runoff, the mean mass of suspended solids, $E(L_r)$, input to the pond is the product of the mean runoff volume and C_0, i.e.

$$E(L_r) = \frac{\phi}{\zeta} e^{-\zeta S_d} C_0 \tag{18}$$

Hence, the long term average percentage removal of suspended solids for a specified settling velocity is

$$\begin{aligned} C_p &= \frac{\overline{R_m}}{E(l_r)} 100\% \\ &= E_d \Omega \left(\frac{1}{\lambda} - T_d\right) \frac{\zeta}{\phi C_1} 100\% \end{aligned} \tag{19}$$

The above equation is designated as the CPED model for extended detention dry ponds. Details of the above derivations can be found in Guo [1992].

3. Statistical Analysis of Wet Ponds

Two types of wet ponds are herein considered: the wet pond with outlet control and the wet pond without outlet control. Wet ponds without outlet control are wet ponds with emergency spillways only. Wet ponds with outlet control are wet ponds with not only emergency spillways but also controlled outlet structures.

3.1 Wet Ponds without outlet Control

The following assumptions are made for this type of pond:

(1) Outflows over the emergency spillway receive no treatment, and the suspended solid

removal mechanism is quiescent settling only.

(2) During the quiescent period, the pond is always at its full storage level, and evaporation and other losses are negligible.

(3) When runoff flows into the pond, it displaces the original contents in the active storage part of the pond. As a result of mixing, dilution and replacement, the suspended solid concentration in the pond after each runoff event is the same as the long term average concentration of the incoming runoff.

At time t within a quiescent settling period, the quiescent sedimentation rate, D_q, of particles with settling velocity V_s can be expressed as follows:

$$D_q = V_s A C(t) \tag{20}$$

where A is the surface area of the pond; and C(t) is the concentration of suspended solids in the pond at time t. Because of continuous settling, the concentration in the pond gradually declines.

In an infinitesimal time step dt, the change of concentration in the pond is

$$dC = -\frac{V_s A C(t)}{S_a} dt, \tag{21}$$

where S_a is the active storage volume of the pond. The active storage is defined as the storage of the pond excluding dead zones, or that part of the storage volume which is usually replaced by incoming runoff.

Integrating the above equation and incorporating the above assumptions, the following result is obtained:

$$C(t) = C_0 e^{-\frac{V_s A}{S_a} t} \tag{22}$$

Substituting Eq. 22 into Eq.20 yields

$$D_q = V_s A C_0 e^{-\frac{V_s A}{S_a} t} \tag{23}$$

Letting the length of the quiescent period be T, the total mass of suspended solid particles removed, R_m, in this quiescent period can be obtained by integrating the above equation as follows:

$$R_m = C_0\left(S_a - S_a e^{-\frac{V_s A}{S_a}T}\right) \qquad \textbf{(24)}$$

For a wet pond without outlet control, the duration of the quiescent period of a loading cycle equals the corresponding rainfall interevent time, b. Knowing that b is exponentially distributed as expressed in Eq.3, the mean mass of suspended solids removed in a loading cycle can be found as follows:

$$\overline{R_m} = \frac{C_0 S_a V_s A}{\psi S_a + V_s A} \qquad \textbf{(25)}$$

Similar to the analysis for extended detention dry ponds, the long term average percentage of suspended solid removal of a wet pond without outlet control can thus be found as

$$C_p = C_c \frac{\zeta}{\phi} e^{\zeta S_d} \frac{V_s S_a A}{\psi S_a + V_s A} 100\% \qquad \textbf{(26)}$$

Where $C_c = A_c/1000$ is the unit conversion factor resulting from the different units used for C_0 in Eq. 25 and Eq. 18. A_c is the catchment area serviced by the pond in m^2. Eq. 26 is designated as the CPW1 model for the wet pond without outlet control. Details of the above derivations can be found in Guo [1992].

3.2 Wet Ponds with Outlet Control

The possible time histories of a loading cycle of a wet pond with outlet control are similar to those of an extended detention dry pond, except that in a wet pond, each cycle begins with the pond at its permanent pool level and returns to the same level. During runoff periods and periods when the water level in the pond is above the permanent pool, the pond is acting in the same manner as an extended detention dry pond. Suspended solids are removed in this period through dynamic settling. In the dry period when the pond is at its permanent pool level, suspended solids are removed through quiescent settling, similar to a wet pond without outlet control.

Since the quiescent period of a wet pond with outlet control is no longer the rainfall interevent time b, but rather $(b+T_d)$, which is usually shorter than b, the results for the wet pond without outlet control cannot be directly used. Inclusion of this difference makes the mathematical derivation extremely difficult. However, accounting for this difference based on the ratio of the mean lengths of quiescent settling times under wet pond without outlet control conditions and wet pond with outlet control conditions should give a reasonable estimate. Thus, the long term percentage removal due to the permanent pool of a wet pond with outlet control can be estimated as follows:

$$C_{pq} = C_c \frac{\zeta}{\phi} e^{\zeta S_d} \frac{V_s S_a A}{\psi S_a + V_s A} \frac{\text{б} + T_d}{\text{б}} 100\% \tag{27}$$

The storage volume between the permanent pool and the spillway crest acts as an extended detention dry pond. However, unlike the extended detention dry pond, particles must settle to the bottom of the permenent pool before they are removed. This is considered by modifying the calculation of E_d. For Example, in Eq. 5, an effective settling velocity V_{se} can be defined as

$$V_{se} = V_s \frac{\frac{S}{A_d}}{\frac{S}{A_d} + \frac{S_p}{A_p}} \tag{28}$$

where S is the storage volume above the permanent pool; A_d is the average surface area of the storage volume above the permanent pool; S_p is the storage volume of the permanent pool; and A_p is the surface area of the permanent pool. Using this effective settling velocity in the calculation of E_d accounts for the increased settling depth due to the permanent pool. In this manner, the result obtained for extended detention dry ponds can be used directly to calculate the dynamic removal in wet ponds with outlet control.

The overall removal of suspended solids by wet ponds with outlet control is the combination of the dynamic removal and the quiescent removal, i.e.

$$\begin{aligned} C_p = \frac{\zeta}{\phi C_1} [C_c \frac{S_{pa} V_s A_{pa}}{V_s A_{pa} + S_{pa} \psi} (1 + \psi T_d) \\ + E_d \Omega (\frac{1}{\lambda} - T_d)] 100\% \end{aligned} \tag{29}$$

where S_{pa} is the active storage volume of the permanent pool (mm), and A_{pa} is the surface area of the active storage volume of the permanent pool (m^2). Eq. 29 is designated as the CPW2 model for wet ponds with outlet control. Details of the above derivations can be found in Guo [1992].

APPENDIX K

SWMM FACILITY LAND USE REQUIREMENTS

Derivations of Land Area Requirements

Land area requirements to implement dry ponds, wet ponds, wetlands and infiltration basins are derived based on assumptions about the preliminary designs of these facilities. Detailed derivations are provided in the following sections.

Wet Ponds and Wetlands

The following configuration was assumed to be indicative of typical design parameters for wet ponds and wetlands:

- bottom of the wet pond/wetland was assumed to be rectangular in shape
- length to width ratio of 3:1
- side slopes of 4:1 within the permanent pool
- side slopes of 4:1 in the extended detention portion of the pond/wetland

Let PV be the permanent pool volume
EV be the extended detention volume
X be the bottom width of the permanent pool

Since

$$
\begin{aligned}
PV &= h_p\,[3X^2 + (X + X + 8h_p)(3X + 3X + 8h_p) + (X + 8h_p)(3X + 8h_p)]/6 \\
&= h_p\,(18X^2 + 96h_pX + 128h_p^2)/6 \\
&= 3h_pX^2 + 16h_p^2X + 128h_p^3/6
\end{aligned}
$$

Therefore

$$X = [256h_p^4 - 12h_p(64h_p/3 - PV)]^{0.5}/(6h_p) - 8h_p/3$$

Let h_e be the depth of the extended detention storage
Since the extended detention volume can be approximated as:

$$
\begin{aligned}
EV &= h_e[(X + 8h_p)(3X + 8h_p) + (X + 8h_p + 10h_e)(3X + 8h_p)]/2 \\
&= 5(3X + 8h_p)h_e + (X + 8h_p)(3X + 8h_p)h_e
\end{aligned}
$$

Therefore

$$h_e = [(X + 8h_p)^2(3X + 8h_p)^2 + 20(3X + 8h_p)EV]^{0.5}/[10(3X + 8h_p)] - (X + 8\,h_p)/10$$

The land area required (LA)is

$$LA = (X + 8h_p + 10h_e)(3X + 8h_p + 10h_e)$$

Dry Ponds and Infiltration Basins

The following configuration was assumed to be indicative of typical design parameters for dry ponds and infiltration basins:

- bottom of the pond/basin was assumed to be rectangular in shape
- length to width ratio of 3:1
- side slopes of 5:1 in the extended detention portion of the pond/basin

Let EV be the extended detention volume
h be the depth of the extended detention storage

Since

$$EV = h[(3X^2 + (X + X + 10h)(3X + 3X + 10h) + (X + 10h)(3X + 10h)]/6$$
$$= h(18X^2 + 120hX + 200h^2)/6$$
$$= 3hX^2 + 20h^2X + 100h^3/3$$

Therefore

$$X = [400h^4 - 12h(100h^3/3 - EV)]^{0.5} / (6h) - 10h/3$$

The land area required (LA) is

$$LA = (X + 10h)(3X + 10h)$$

APPENDIX L

PERFORATED PIPE EXFILTRATION

Perforated Pipe Exfiltration

Recent work (Wisner and Associates, 1993) has indicated that the exfiltration can be reasonably modelled using the orifice equation with a variable orifice coefficient. The orifice coefficient varies from 0 to a maximum of 0.63 and is dependent on both the perforation size and the depth of flow.

Wisner and Associates determined that for 7.9 mm and 12.7 mm diameter perforations the heads at which the maximum discharge coefficient were obtained were 55 mm and 180 mm, respectively. Above these depths a constant discharge coefficient of 0.63 would apply. Equation L.1 provides an estimate of the discharge coefficient using an elliptical distribution (Wisner and Associates, 1993).

$$C = 0.63 \sqrt{1 - (1 - \frac{H}{H_{max}})^2}$$

Equation L.1 Exfiltration Discharge Coefficient

where C = orifice discharge coefficient and head H
H = head over orifice (mm)
H_{max} = head above which the discharge coefficient is constant (mm) (ie. 55 mm for 7.9 mm diameter perforations and 180 mm for 12.7 mm perforations)

If the head above the orifice is greater than H_{max} the discharge coefficient is constant at 0.63. Once the discharge coefficient is determined the flow from the pipe into the exfiltration storage can be calculated using the standard orifice equation (Equation L.2).

$$Q = C\ A\ \sqrt{2\ g\ H}$$

Equation L.2 Orifice Equation

where Q = exfiltration flowrate (m^3/s)
C = orifice discharge coefficient
A = area of the perforations (m^2)
g = gravity acceleration (9.81 m/s^2)
H = head over perforations (m)

These equations (L.1 and L.2) must be calculated in a dynamic continuous simulation since the flow through the pipe, and hence depth over the orifices, decreases as a result of exfiltration. Steady state calculations using Equation L.1 and L.2 yield erroneous results.

Therefore the results for the Wisner study were reviewed to derive an empirical relationship between exfiltration flow in a perforated pipe and flow through the pipe based on the number and size of perforations, size of pipe, and slope of pipe.

Based on a review of the laboratory results the following empirical derivation was used to determine Equation 3.16.

@ 2% slope and 12.7 mm diameter perforations

$0.5Q = AC_1 + SC_2 + b$

$0.5Q = 0.02C_1 + 2C_2 + b$ (1)

@ 2% slope and 7.9 mm diameter perforations

$0.33Q = AC_1 + SC_2 + b$

$0.33Q = 0.008C_1 + 2C_2 + b$ (2)

Subtracting equation 2 from 1 and rearranging

$C_1 = 14.2$ or approximately 15

@ 1% slope and 12.7 mm diameter perforations

$0.58Q = AC_1 + SC_2 + b$

$0.58Q = 0.02C_1 + C_2 + b$ (3)

Subtracting equation 1 from 3 and rearranging

$C_2 = -0.08$

@ 1% slope and 7.9 mm diameter perforations

$0.375Q = AC_1 + SC_2 + b$

$0.375Q = 0.008C_1 + C_2 + b$ (4)

Subtracting equation 2 from 4 and rearranging

$C_2 = -0.045$

Therefore C_2 is approximately -0.06 and the following relationship was determined :

Q_{exf} = (15 A - 0.06S +b) Q_{inf}

For 2% slope and 12.7 mm perforations	b = 0.32
For 1% slope and 12.7 mm perforations	b = 0.34
For 2% slope and 7.9 mm perforations	b = 0.33
For 1% slope and 7.9 mm perforations	b = 0.32

The average b value is 0.33

Therefore a simple approximation for the exfiltration flow through a perforated pipe is :

$$Q_{exf} = (15A - 0.06S + 0.33)\, Q_{inf} \qquad (5)$$

where
A = area of perforations per metre length of pipe (m^2/m)
S = slope of the perforated pipe (%)
Q_{inf}= flow through the pipe (m^3/s)
Q_{exf}= flow through the pipe perforations (m^3/s)

The exfiltration flows based on Equation 5 were compared with the laboratory results from the Wisner study. The results are shown on the accompanying Figures L.1 through L.4. The empirical relationship reasonably approximates the exfiltration-pipe flow relationship in all four cases.

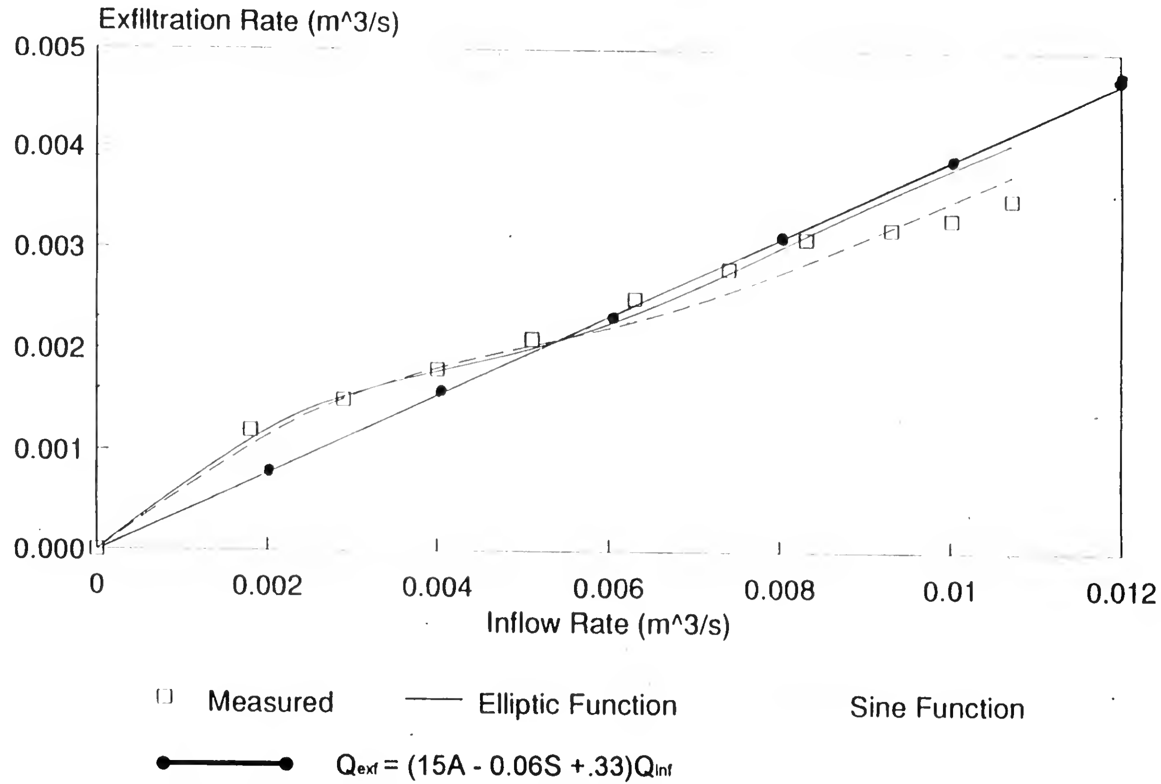


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FIGURE L.1 COMPUTED EXFILTRATION RATES BASED ON A VARIABLE ORIFICE DISCHARGE COEFFICIENT. (ORIFICE SIZE = 7.9 mm, PIPE SLOPE = 1%)

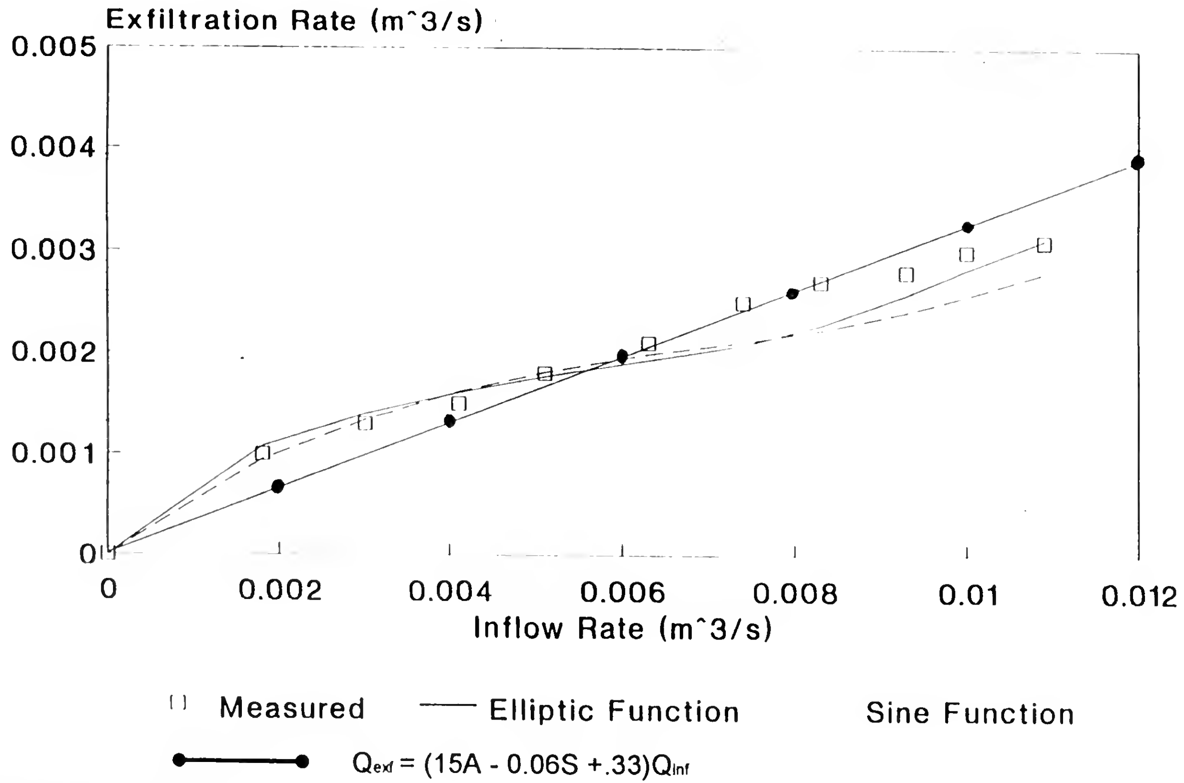

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FIGURE L.2 COMPUTED EXFILTRATION RATES BASED ON A VARIABLE ORIFICE DISCHARGE COEFFICIENT, (ORIFICE SIZE = 7.9 mm, PIPE SLOPE = 2%)

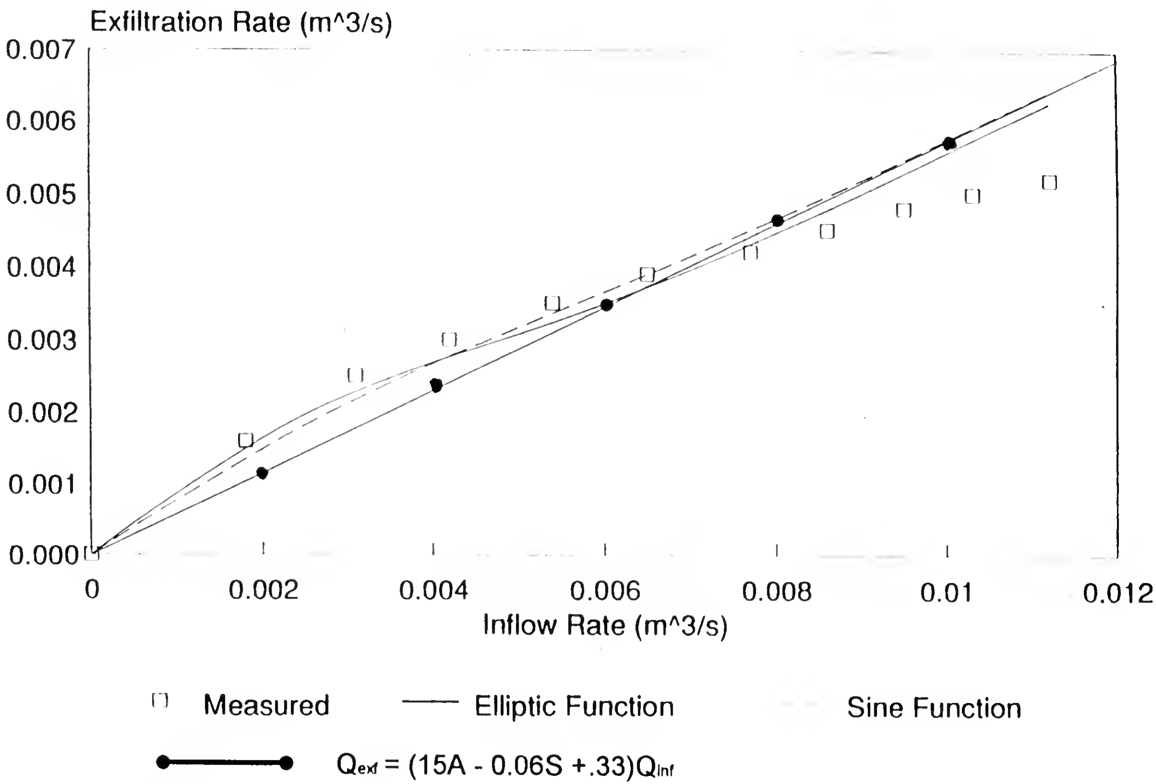

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FIGURE L.3 COMPUTED EXFILTRATION RATES BASED ON A VARIABLE ORIFICE DISCHARGE COEFFICIENT, (ORIFICE SIZE = 12.7 mm, PIPE SLOPE = 1%)

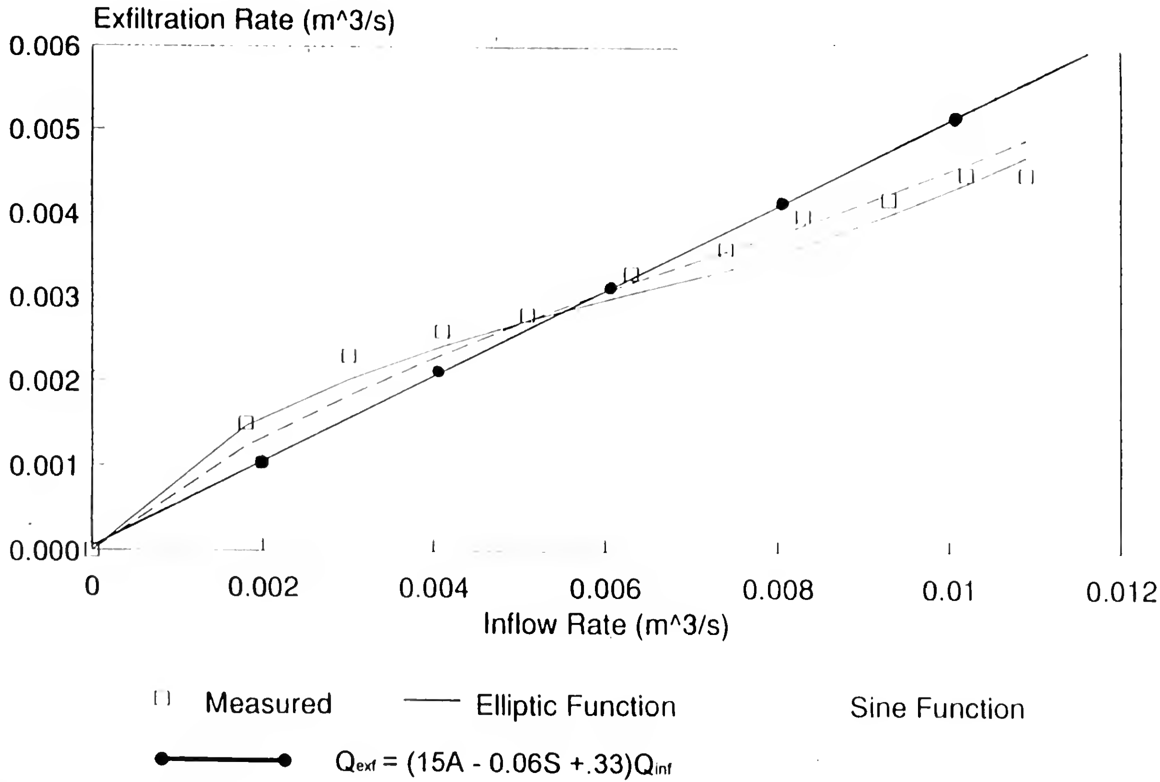

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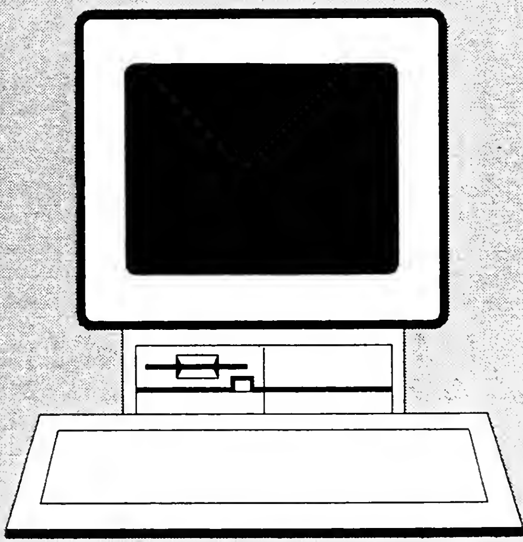
FIGURE L.4 COMPUTED EXFILTRATION RATES BASED ON A VARIABLE ORIFICE DISCHARGE COEFFICIENT, (ORIFICE SIZE = 12.7 mm, PIPE SLOPE = 2%)

Stormwater Management Practices Computerized Information System



Computerized Information System

The manual is available as a computerized information system. The information system provides a synopsis of the material covered in the manual (ie. not all of the information in this manual has been incorporated in the information system).

Intent and Scope

The information system was conceived and designed to educate and inform a broad spectrum of people regarding stormwater management from the interested public to professionals in the stormwater management field. A computer media was used since computers can combine graphical and textural information into an interactive user-friendly presentation package.

The information system illustrates the treatment train approach to stormwater management similar to the hardcopy manual. The information system (IS) educates the user with respect to:

- watershed and subwatershed planning and the linkages to urban development subdivision/site planning and stormwater management practices
- subdivision/site planning techniques and their importance to achieving stormwater management objectives
- individual stormwater management practices including stormwater lot level controls, stormwater conveyance controls, and end-of-pipe stormwater management facilities
- the selection and design of a stormwater management practices in the absence of watershed / subwatershed planning
- a reviewer's checklist to ensure that the major concerns/parameters associated with a stormwater management practice have been addressed

System Requirements

The information system was written in an expert system shell. Although this version of the IS does not allow the user to actually design stormwater management plans, subsequent revisions/upgrades/add-ons will take advantage of the calculation engine provided in the expert system software. The software allows forward and backward chaining and can interface with database information making it a versatile tool to build expert system design modules.

The information system is a Windows program. The user must have Windows on his/her computer (or on a network) to use this product. The user should have a 386 processor (as a minimum) to run the information system. In addition, the computer must have at least 4 Mb of RAM and 30 Mb of free hard disk space. Installation instructions are provided on the disks in the README.DOC file.